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AN INVESTIGATION OF THE EFFECTIVENESS OF STIFFENERS ON SHEAR-RESISTANT PLATE-GIRDER WEBS

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SUMMARY

The results of 60 different tests on 2 aluminum alloy 17S-T plate girders are presented to show the influence of size and spacing of stiffeners upon the buckling characteristics of shear-resistant webs within the elastic range. It is demonstrated that stiffeners increase the stability of a web by retarding the formation of buckles and by providing partial edge restraint to the subdivided panels. An empirical method of proportioning stiffeners is proposed which recognizes both of these stiffener functions, and comparisons are made with design procedures based upon theoretical considerations of the buckling problem. Also, some experimental data are provided to show the effect of stiffener size and spacing upon ultimate web strengths.

INTRODUCTION

Although stiffeners have been used for many years to prevent shear buckling in plate girders of structural steel, apparently little progress has been made in placing the design of stiffeners for this class of structure upon a rational basis. The specifications for steel railway bridges adopted by the American Railway Engineering Association in 1910 required that the width of outstanding leg on intermediate stiffeners should be not less than one-thirtieth of the depth of the girder plus 2 inches, and this same requirement is incorporated in the 1938 specifications. In plate girders with a uniform depth, no provision is made for varying the size of stiffener as stiffener spacings are varied; this procedure is obviously essential for a balanced design.

The increasing emphasis being placed upon the use of more accurate methods in the design of light-weight structures, particularly those for aircraft, requires some consideration of the stiffener problem. In reference 1 (p. 418) Timoshenko gives some data pertaining to the flexural rigidity of stiffeners required to stiffen panels of different proportions. (See also reference 2.) Although theoretically the stiffener size increases with the number of stiffeners used on any given web, only cases involving one and two stiffeners have been considered. In the application of these results to practical design, Timoshenko assumes a required stiffness no more than double that indicated by the theory for one stiffener.

The empirical formula for stiffener size given in reference 3 is based upon a proposal by E. Chwalla found in reference 4. (See also reference 5.) This solution appears to be somewhat more suited for design than the analysis of Timoshenko because it covers any number of stiffeners.

The value of any solution on the basis of design depends upon how closely it predicts actual behavior. Any attempt to correlate tests results and the theory for shear buckling in stiffened plate-girder webs, of course, involves a number of complicating factors. Probably of foremost importance is the fact that definite critical buckling loads usually cannot be experimentally determined, either for the individual web panels between stiffeners or for the stiffened panels as a whole. Because of eccentricities of loading, lateral deflections may occur in both stiffeners and web from the early stages of a test and no point that might be called critical, or might serve as a basis for judging the effectiveness of a given stiffener, will be observed. Complete failure of a web as a shear-resistant member usually cannot occur because of the redistribution of stress that accompanies large deflections; hence the significance of a critical buckling load in shear, even if it could be definitely determined, is somewhat questionable.

The objects of this investigation were: (1) to determine experimentally the influence of size and spacing of stiffeners upon the buckling characteristics of shear-resistant plate-girder webs within the elastic range; (2) to evaluate, as far as possible from the test results, certain methods of stiffener design that have been proposed; and (3) to obtain some information on the influence of stiffener size and spacing upon ultimate web strengths.

DESCRIPTIONS OF SPECIMENS

All the stiffener tests were made on two plate girders, designated specimens A and B, which were fabricated from 17S-T aluminum-alloy plates, angles, and rivets. Figures 1 and 2 show the structural details of the two girders and give the principal stress and deflection factors for the type of loading used.

Table I summarizes the results of mechanical property tests on the plate and angle materials. The tension tests were made according to the method of reference 6; the compression tests were made by the single-thickness method described in reference 7. All strength values are considerably above the guaranteed minimums (see reference 8) for 17S-T, although they are not outside the range of properties frequently obtained on sheet and extruded forms of this alloy. From the values of yield strength obtained for the webs in both tension and compression, the yield strength in shear, which is of particular interest for the purpose of these tests, was estimated to be in the vicinity of 24,000 pounds per square inch.

Although the choice of web proportions was quite arbitrary, an attempt was made to provide specimens in which different stiffener spacings would give a wide range of buckling resistances within the elastic strength of the web material. The flanges were proportioned to provide comparatively low ratios of maximum bending to shear stress in order to minimize the effect of bending upon the buckling of the webs. This feature of the design is emphasized by the fact that the ratios of shear to bending deflection at the center of the spans under central concentrated loads were computed to be approximately 2:1 for both girders.

Figure 3 shows the different stiffener spacings investigated and gives the theoretical buckling loads and corresponding average shear stresses for the subdivided panels, assuming simply supported edges. (See reference, 9, p. 60.) Panel widths were assumed to be equal to the distances center to center of intermediate stiffeners.

Eight different sizes of intermediate stiffener angles of 17S-T, ranging from 1/2 by 1/2 by 1/16 inch to $1\frac{3}{4}$ by $1\frac{1}{4}$ inches by 1/4 inch, were provided, although all sizes were not used for each spacing indicated in figure

3. Figure 4 shows the details of these stiffener connections.

PROCEDURE

Figure 5 shows a typical test set-up in the 300,000-pound-capacity Amsler testing machine. Central concentrated loads were applied on both girders, the end reactions being carried through aluminum-alloy plate and shelf-angle supports bolted to the end stiffeners. Roller-bearing supports were used as indicated to permit free movement at the ends of the span, resulting from lengthening of the bottom or tension flanges.

Measurements of lateral deflection, which were used to indicate the buckling characteristics of the webs and the stiffeners, were made by means of the apparatus shown in figure 6. The use of a dial indicator, graduated in thousandths of an inch, between the webs of the girders and a reference bar held against the top and bottom flanges made possible the rapid determination of deflections within 0.001 or 0.002 inch. Readings were taken at seven different stations over the clear depth of each web on sections spaced 2 to 4 inches along the length of the girders.

In order to determine experimentally the effect of a number of different sizes and spacings of stiffeners upon the behavior of a single web, it was necessary to produce fairly definite buckle patterns for each case without exceeding the elastic strength of the material. For cases involving relatively few stiffeners this requirement was easily met although, as the number of stiffeners increased, it became increasingly difficult to obtain the desired buckle patterns without producing permanent sets. The theoretical buckling loads for an assumed condition of simply supported edges (see fig. 3) were used as a guide in the selection of safe loads, although in no cases were the average shear stresses allowed to exceed 20,000 pounds per square inch, or a value slightly below the shear yield strength estimated for the web material used. Loads were applied in increments up to the maximum value selected for each case, after which permanent-set measurements were made.

Figure 3 indicates the order in which the different stiffener spacings were investigated on each girder. The first tests were made on the largest two panels without stiffeners, labeled "First Test"; later tests involved 1, 2, 3, 5, or 7 stiffeners, labeled "Series I, Series II," and so forth. Table II indicates the sizes of stiffeners included in each series. The smallest stiffeners for each spacing were selected as far as possible from theoretical requirements (reference 1, p. 418), while the largest sizes had a stiffness many times the theoretical values. The order of tests was generally according to stiffener size, starting first with the smallest single angle to be investigated for a given case and proceeding through a series of 4 to 10 different tests to a pair of the larger angles. The tests for any particular spacing were stopped when a pair of stiffeners was obtained that showed relatively little lateral deflection as compared with the deflection found for the web panels, provided such a condition could be obtained with the stiffeners available and without exceeding the imposed limit of 20,000 pounds per square inch for average shear stress.

Lateral-deflection measurements in each test were limited to the half of the span where the stiffener sizes were varied (series I, II, III, etc.), which, as is indicated in figure 3, alternated from side to side with each change of stiffener spacing. The sizes and spacings of stiffeners used on the opposite half of the girders for each series (fig. 3) generally produced a more stable web condition than that to be investigated; hence deflection readings throughout the length of the span in each test were not deemed necessary.

The method used in determining the flexural rigidity of single-angle stiffeners differs from the methods that have been proposed by other investigators. Instead of using the moment of inertia for an angle alone, about the face of the web to which it was attached, an effective width of web equal to 25 percent of the clear depth was assumed to act with each stiffener. The justification for such a procedure regarding effective widths is based upon observations made in a previous investigation. (See reference 10.) The use of an axis in the face of the web, which recognizes the stiffening influence of the web, seems somewhat inconsistent in that it implies a different effective width for each size of stiffener. For a 1/2- by 1/2- by 1/16-inch angle on a 1/8-inch web, for example, an effective width of web of 1 inch is sufficient to shift the neutral axis for the combined section to the face of the web. For a 3/4- by 3/4- by 3/16-inch angle, however, approximately 8 inches of effective width are required for a corresponding change in the position of the neutral axis.

Table III shows a comparison of moments of inertia for all sizes of angle determined by the two methods. For the small single angles the values obtained when an effective width equal to 25 percent of the depth was assumed were larger than those computed for the angles alone about an axis in the face of the web; for the larger angles this relative position was reversed. Although the differences between moments of inertia computed by the two methods are in most cases not significant, the effective-width method seems to provide a more logical basis for the interpretation of test results. Effective widths of web were neglected in computing moments of inertia for the double-angle stiffeners, where the neutral axis from symmetry was in the middle plane of the web.

At the conclusion of the tests to determine the effectiveness of different sizes and spacings of stiffeners within the elastic range, both girders were tested to failure. (Figs. 9 and 10, to be discussed later, show the conditions investigated.) In these final loadings, the lateral-deflection measurements were supplemented by 2-inch Berry strain-gage readings on the flanges and stiffeners. (Figs. 24 to 29 show the location of the gage lines used.) Vertical deflections at the center of the spans were also determined, using mirrored scales attached to the webs, midway between flanges, and fine wires stretched between the ends of the spans.

DISCUSSION OF RESULTS

Analysis of Lateral Deflections

An analysis of the buckling phenomena observed in this investigation involves a study of load-lateral deflection data obtained from 60 different tests. Although no attempt has been made to show the results of all measurements, figures 7 to 10 show typical load-deflection relations and buckle patterns for different sizes and combinations of stiffeners.

Figures 11 to 18 show average load-lateral deflection curves for the web panels and stiffeners in all tests. The web deflections are the average of the maximum measured values found midway between stiffeners, which were also the maximum values for each panel in most cases. The stiffener deflections are the average of the maximum values measured for each stiffener. Although considerable variation was found in some cases between the deflections of supposedly like panels and stiffeners, average rather than individual maximum values were believed to provide the most satisfactory basis for a general interpretation of the test results. The influence of different amounts of bending upon the shear-buckling tendencies of a series of like panels was apparently negligible. Table II, which gives a summary of all but the ultimate load tests, indicates the maximum range of web and stiffener deflections observed.

From the nature of the load-deflection curves shown in figures 11 to 18, it seems quite evident that a definite value cannot be experimentally determined for the flexural rigidity of stiffeners required to stiffen panels of given proportions, such as might be obtained by application of the buckling theory. The first difficulty encountered is in the determination of critical loads or the relative buckling resistances for the different sizes of panel from which some measure of stiffener effectiveness might be obtained. Although most of the curves in figures 11, 14, 15, and 16 show a fairly pronounced knee, which is believed to be indicative of some buckling phenomena, a quantitative comparison of these results is obviously difficult. In curves of the type shown in figures 12, 13, 17, and 18 the change in the rate of deflection is so gradual that buckling apparently was not involved. An analysis of these average load-deflection data by the Southwell method (reference 1, p. 177) failed, moreover, to provide a generally satisfactory basis for the selection of critical buckling loads.

In spite of the questionable status of the buckling involved in these tests, the results indicated fairly consistently that the average lateral deflections of the web panels decreased with increasing sizes of stiffener. Where such a behavior was observed, it seems reasonable to assume that the buckling resistance of the web panels had been increased by increasing the size of stiffener. This increase may be attributed both to the effect of edge restraint along the boundaries of the panels and to the increased effectiveness of the larger stiffeners in confining buckling to the web. The buckling theory previously referred to assumes that the stiffeners need support the subdivided web panels only until the critical load for a condition of simply supported edges is developed, after which general buckling may occur. It appears from these tests that, although a given size of stiffener may apparently meet this requirement, a larger size may result in a greater buckling resistance in the web.

From the load-deflection curves for the largest panels tested without stiffeners, there seems little question that the actual buckling loads were considerably above the theoretical values for a condition of simply supported edges. The curves shown in figure 7 for specimen A are believed to be as satisfactory for determining experimental buckling loads as any obtained and indicate a critical value in the vicinity of 40,000 pounds. From the ratio of the theoretical buckling values for this size of panel for fixed and simply supported edges (see reference 11), a load of 40,000 pounds corresponds to an edge fixity of about 70 percent. The estimated buckling load of 20,000 pounds for the unstiffened 24- by 48-inch web panel of specimen B corresponds to a fixity of almost 84 percent. The difference in apparent edge restraint for the two specimens is of the order expected in view of the fact that different sizes of flange angle were used on webs of the same thickness.

Although no attempt was made to estimate buckling loads for the tests involving intermediate stiffeners, it seems reasonable to assume that edge restraint also had a significant bearing upon the deflections observed for these cases. In order to permit some estimate of this effect, theoretical buckling loads for a condition of simply supported edges are indicated on the load-deflection curves in figures 7 to 10 and in figures 11 to 18.

In a few tests involving a close spacing of stiffeners, loads were applied which produced accidental permanent sets sufficient to influence the buckling characteristics of the webs and stiffeners in all subsequent loadings. In the case of specimen A shown in figure 12, for example, the first test was made on an intermediate size of stiffener (test 3). Both larger and smaller sizes were left to be investigated later. Although the loads applied in this first test did not involve an average shear stress greater than 17,500 pounds per square inch, the permanent sets measured in the web were larger than the values found in any previous case. As a result, the load-deflection relations observed for both web and stiffeners in all subsequent tests indicated the effect of some eccentricity of loading.

Table II gives a summary of the maximum permanent sets measured for the webs and stiffeners in all tests. In most cases these values do not appear large enough to indicate any significant departure from the range of elastic action. Permanent sets of 0.015 inch or greater were found in the web in only three tests and these involved average shear stresses ranging from 17,300 to 18,700 pounds per square inch, which were undoubtedly above the elastic range of the web material.

Figures 19 and 20 show the results of an attempt to reduce all tests to a basis of comparison where some appraisal of the effect of size and spacing of stiffeners and the effect of edge restraint might be made. Since definite values of buckling load could not be experimentally determined, test loads corresponding to certain arbitrary values of lateral deflection were selected from figures 11 to 18 to indicate relative buckling resistances. Loads corresponding to average maximum deflections of 0.060 inch in the web and 0.020 inch in the stiffeners were selected for comparison with the theoretical buckling loads for the web panels assuming simply supported edges. These load ratios are plotted as ordinates in the figures. It appears significant that, for some cases at least, a lateral deflection of 0.060 inch in the web was within the range of deflections where buckling occurred, according to analyses of the load-deflection data made by the Southwell method. Such an arbitrary value of deflection does not, of course, imply the same degree of buckling for all the different sizes of panels investigated, which is admittedly an objectionable feature of the method of comparison used. Eccentricities of loading that may have had a negligible effect in panels having a low buckling resistance may have accounted for the entire deflection of 0.060 inch, where high buckling resistances were involved. An average deflection of 0.020 inch was used for the stiffeners, both because it was small and because it was one value within the range of values measured for most of the sizes investigated.

The abscissas in figures 19 and 20 are ratios of the flexural rigidity (EI) of one stiffener to that for a web panel between stiffeners, defined here as the ratio λ. The moments of inertia used for the stiffeners in computing these ratios are shown on the load-deflection curves in figures 11 to 18. As previously indicated, the values for the single-angle stiffeners include an effective width of web equal to 25 percent of the clear depth. Although the deflections of the stiffeners appear reasonably consistent in most cases with the moments of inertia computed, the relative positions of the load-deflection curves can hardly be used to demonstrate the correctness of the effective-width method over that in which the moments of inertia for single angles are computed about the face of the web in contact with the stiffeners. The moments of inertia of the web panels between stiffeners were computed from the relation:

$$I = \frac{bt^3}{12}$$

where

I moment of inertia, inch^4

b stiffener spacing, inch

t web thickness, inch

Several observations may be made from figure 19, showing the influence of stiffener size upon web deflections, which appear significant from the standpoint of design. The ratios of the test loads corresponding to an average lateral deflection of 0.060 inch in the web panels to the theoretical buckling values for the case of simply supported edges are shown to increase with increasing size of stiffener for any given proportions of panel. Such a result not only indicates the extent to which stiffener size may influence the buckling resistance of the webs but also suggests that in no instance were the tests carried far enough to obtain the maximum possible web efficiencies. For values of λ greater than those shown, the load ratio should presumably approach a constant value. As the proportions of the panels were changed, however, and a closer spacing of stiffeners used, the ratios of the test to the theoretical buckling loads decreased. For example, the values obtained for specimen B having only one stiffener $(b/d = 1)$ correspond to an edge condition ranging from 30 to almost 100 percent fixed. The ratio of buckling loads for fixed edges to simply supported edges is assumed equal to 1.68 for all sizes of panel, which is the theoretical ratio for infinitely long plates. (See reference 1, p. 362, and reference 11.) For the case of seven stiffeners $(b/d = 1/4)$, the ratios correspond to test loads less than the theoretical values for panels with simply supported edges. In other words, the effectiveness of the stiffeners, as measured by a constant value of web deflection, decreased as the stresses corresponding to the computed buckling loads increased. The relative position of the load-stiffness ratio diagrams for specimens A and B, for cases involving the same proportions of panel with one or two stiffeners, corresponds to the position that would be expected from the different degrees of edge fixity indicated in the tests of the unstiffened panels.

The curves in figure 20, showing the influence of stiffener size upon stiffener deflection, indicate about the same relative behavior for panels of different proportions as shown in figure 19. The shape of the curves is fundamentally different, however, in that the load ratios approach an infinite rather than a constant value as the size of stiffener is increased. Stiffener deflections may approach zero; whereas deflections for the web cannot be reduced below the deflections accompanying buckling for edges completely fixed.

Proposed Basis for Stiffener Design

In the selection of stiffener sizes suitable for design from the results of these tests, an attempt was made to recognize as far as possible the principal characteristics of behavior noted in the foregoing figures. The degree of edge fixity obtained for any case is, of course, not known, and various interpretations may be placed upon the significance of the load-deflection curves shown in figures 11 to 18 with respect to this factor. For the tests in which a fairly definite buckling action was apparent within the elastic range, it is believed that an assumption of 50-percent edge fixity, which involves loads approximately 34 percent greater than the theoretical buckling values for panels with simply supported edges, may well be made as a basis for selecting relative proportions of webs and stiffeners. For the tests in which web buckling was not so evident, an average shear stress of 16,000 pounds per square inch appeared to mark the approximate limit of elastic action, and loads corresponding to this stress were assumed to be equally significant from the standpoint of stiffener design. Figures 11 to 18 show the position of the lower or critical value of these two arbitrary design-load limits with respect to the average web and stiffener deflections measured for each size of panel.

Some arbitrary limits on stiffener deflections were also necessary because none of the stiffeners investigated remained straight under the design loads selected, and stiffener effectiveness could be determined only on a relative basis. It is believed that a stiffener which shows essentially the same load-deflection characteristics as the web that it supports (and there are numerous such cases indicated in figs. 11 to 18) is not adequate, regardless of the loading for which buckling may seem to occur. Two arbitrary deflection requirements were therefore imposed: (1) that the stiffener deflection not exceed 0.020 inch for the design loading assumed, and (2) that the stiffener deflection not exceed 25 percent of the smallest average web deflection observed for this loading.

The moments of inertia required of stiffeners to meet the foregoing conditions may be estimated from the values of moment of inertia indicated on the load-deflection curves in figures 11 to 18. For each proportion of panel investigated, one value for stiffener moment of inertia was obtained. Although the buckling theory indicates that the number of stiffeners used in providing panels of given proportions has a significant bearing upon the flexural rigidity required for each stiffener, the limited scope of these tests did not make possible a consideration of this factor. In order to make the results obtained generally applicable to design, ratios of the flexural rigidity of each selected stiffener to the flexural rigidity for the corresponding web panels were determined and plotted against proportions of panel, as shown in figure 21. The relationship obtained may be expressed approximately as:

$$\lambda = \frac{14}{\left(\frac{b}{d}\right)^3}$$

where

λ ratio of flexural rigidity of one stiffener to flexrual rigidity of web panel between adjacent stiffeners

d clear depth of web, inch

Figures 11 to 18 show estimated load-lateral deflection curves for stiffeners proportioned by means of the foregoing empirical formula. The relative position of these curves with respect to those determined from the tests is, of course, only approximate, since the measured deflections themselves were not always consistent with the moments of inertia involved.

Figures 19 and 20 provide a basis for evaluating the proposed design formula in terms of observed lateral deflections and theoretical buckling loads for the web panels. The stiffener sizes computed for every case investigated were sufficient to develop loads from 1 to $1\frac{1}{2}$ times the theoretical values for a condition of simply supported edges without exceeding an average web deflection of 0.060 inch, or a value less than one-half the web thickness. These load ratios correspond to edge-fixity factors ranging from zero for the closest stiffener spacing (b/d = 1/4) to 73 percent for the widest stiffener spacing (b/d = 1).

Comparison of Stiffener Design Methods

For purposes of comparison, the moments of inertia of stiffeners computed by the other two methods previously referred to are also included in figures 11 to 18. In the first method the moments of inertia were determined from reference 3, where

$$I = \frac{(0.1 + 0.02N)t^3d}{\beta^4}(\beta^2 + 0.625)$$

but not to exceed

$$\frac{0.2\ t^3d}{\beta^4}(\beta^2 + 0.625)$$

where

N number of stiffeners

d over-all depth of web, inch

β ratio of stiffener spacing to over-all depth of web (Use $\beta = 0.4$ for all ratios less than 0.4.)

In the second method the moments of inertia were determined from the theoretical treatment of the stiffener problem given in reference 1 (p. 418), where ratios of flexural rigidity λ, as previously defined, are given for cases of one or two stiffeners on panels of different proportions. The moments of inertia selected for design on the basis of the tests were in most cases considerably greater than those obtained by either of the other two methods. No attempt was made to apply the theory to cases involving more than two stiffeners.

In the comparison of these different methods of computing moments of inertia for stiffeners, it should be pointed out that the empirical formula proposed from the tests and the theoretical solution given by Timoshenko in reference 1 involve ratios of stiffener spacing to clear depth of web; whereas the formula given in reference 3 involves ratios of stiffener spacing to over-all depth. The significance of the over-all depth dimension from the standpoint of web buckling is not obvious unless a constant ratio of clear to over-all depth is assumed. It appears that the design of the flange for a particular girder might be varied in such a manner as to influence the buckling resistance of the web appreciably without changing the over-all depth and hence the size of stiffener required to prevent such buckling.

Another feature of the formula given in reference 3 to which attention is called is the indication of constant size of stiffener for cases involving five or more stiffeners, where the ratio of stiffener spacing to over-all depth is 0.4 or less. Unfortunately, the deflections shown in figures 17 and 18 for tests that meet these conditions do not permit any conclusion regarding this limit on maximum stiffener size. From the standpoint of elastic stability, however, it would seem that for a given depth and thickness of web the size of stiffener should always increase as the stiffener spacing decreases; otherwise the resistance to general buckling would fall below the resistance for the subdivided panels.

Table IV presents a further comparison of these stiffener design methods applied to a plate girder having proportions far outside the range investigated. The example of plate-girder design in table IV is taken from reference 5. As in most of the cases previously considered, the flexural rigidities required by the empirical method proposed are the highest. The maximum sizes proposed for the double-angle stiffeners, however, are no larger than those required by current specifications for designs in steel. (See reference 12.) It will be noted that the same size of stiffener is required by the Moisseiff-Lienhard method of reference 3 for two of the three stiffener spacings considered; whereas the method proposed provides a different size for each spacing, which seems to be a more logical procedure. For the cases shown, it appears that the method used in computing moments of inertia for the single bulb-angle stiffeners, whether based upon the assumption of a definite effective width of web (I_{4-4}) or upon the assumption that bending in the stiffener is produced about an axis in the face of the web (I_{3-3}), may not be important as far as the actual size of the angle used is concerned. It should be recognized, however, that for certain proportions of web and stiffener, moments of inertia computed about the face of the web in contact with the stiffener (I_{3-3}) may be higher than those obtainable from any reasonable assumption regarding effective widths. In the case of the $6\frac{1}{2}$- by 3- by 3/8-inch and 6- by 3- by 5/16-inch bulb angles given in the table, for example, the values of I_{3-3} correspond to effective widths over twice the stiffener spacing or the maximum width available for each stiffener. One of the most significant observations to be made from the stiffener elements given in the table is that the single bulb-angle stiffeners are much more effective, from the standpoint of weight-stiffness ratios, than the conventional double-angle type of stiffener.

Ultimate-Load Tests

Although stiffeners proportioned by the method proposed are seemingly adequate for shear stresses within the elastic range, their ultimate resistance to buckling is also important from consideration of design. Ultimate-load tests on the two girders used throughout the investigation have provided an opportunity to obtain a few data on this aspect of the stiffener problem. Figures 9 and 10 show the sizes and the spacings of stiffeners used in the ultimate-load tests. The flexural rigidity of the stiffeners on the left half, where the closest spacings were used, was chosen to agree approximately with the requirements of the proposed design formula. The same sizes were also used for the wider spacings on the right half to provide an extra margin of stiffener rigidity (46 percent for specimen A and 86 percent for specimen B) to offset in some measure the differences in buckling resistance for the two sizes of web panel.

Table V gives the results of the ultimate-load tests with the corresponding computed average shear and maximum bending stresses. The shear stresses developed in the webs of both girders were in the vicinity of the shear yield strength estimated for the web material, which is generally assumed to be the design limit for shear-resistant web action. The strengths obtained in these tests, therefore, were as high as could be reasonably expected.

Essentially the same type of failure was obtained in both girders. The severe buckling action produced on the weak half of the webs eventually broke the machine-screw connections holding the stiffeners, so that collapse and fracture of the webs immediately followed. The stiffeners on specimen A were badly bent before failure of the connections, but those on specimen B apparently were undamaged except for a somewhat battered condition at the ends where they were pinched between the flange angles. Figures 22 and 23 show the nature of the failures obtained. In specimen A, the wide diagonal-tension fracture produced in the web passed through one of the holes for the stiffener connections, which presumably constituted a "stress raiser." In specimen B, the concentration of tensile stress at the upper corner fractured the web and sheared the end-flange rivet.

Failure in the stiffener connections was not expected in these tests, although the weakness of such details must be recognized as a possibility in design. From the large distortions produced in the stiffeners on the right half of specimen A before failure occurred, it appears that about the maximum possible degree of effectiveness was obtained from these stiffeners, and there is little reason to question the adequacy of the connections. The connections used for the stiffeners on specimen B are admittedly smaller than would have been used if this detail had not been carried over from previous tests involving smaller angles. The use of stronger connections undoubtedly would have increased the load-carrying capacity of the web; although the method to be used in designing such details, other than maintaining reasonable proportions, is not apparent. Even though the connections used on specimen B were not adequate to develop the full flexural rigidity of the stiffeners, their shortcomings in this particular test are not considered serious in view of the high average shear stress developed.

The lateral deflections shown in figures 9 and 10 and the condition of the girders after failure shown in figures 5 and 22 indicate that the stiffeners used on the left half of both girders were adequate to develop the full strength of the webs as shear-resistant members. It is obviously not possible to say what margin of strength these stiffeners may have had against ultimate collapse as tension-field action became more pronounced.

The fact that stresses in the vicinity of the shear yield strength of the material were developed, however, without any signs of stiffener weakness appears to be a significant observation from the standpoint of design. For loads within a few percent of the maximum applied, none of the stiffener deflections on the left half exceeded 0.035 inch. The theoretical buckling loads for the web panels were also near the shear yield strength; appreciable web deflections were therefore not produced until stresses in excess of about 20,000 pounds per square inch stress were imposed.

The lateral deflections produced on the right half of the girders, where web failures ultimately occurred, were of much greater magnitude than those found on the left half. The buckle patterns shown in figures 9 and 10 for loads near the ultimate load indicate two quite different types of action. In specimen A the wave formation was continuous across the stiffeners and this pattern, as shown in figure 22, was not changed appreciably by failure involving some degree of tension-field action. In specimen B the stiffeners were sufficiently rigid to confine buckling almost entirely to the web panels and three or more half-waves were produced in each. As soon as the stiffeners were broken off, however, a typical tension-field buckle pattern was produced, as shown in figure 23.

In view of the fact that the stiffeners used on the right half of both girders had flexural rigidities somewhat greater than the rigidity required by the proposed formula, it is only possible to estimate the adequacy of the formula for these particular cases. There is apparently little question concerning the stiffeners on specimen B because only small lateral deflections were observed and a maximum shear stress was developed which was greater than the yield strength of the material and approximately 90 percent greater than the theoretical buckling stress for the web panels. A decrease of 54 percent in the flexural rigidity of the stiffeners, in accordance with the proposed method, would not, it is believed, seriously impair the strength of the web.

In specimen A large stiffener deflections were not observed until loads corresponding to an average shear stress of about 20,000 pounds per square inch were imposed. Under such conditions, plastic yielding of the web would be expected and the accompanying loss in buckling resistance should result in some deflection of the stiffeners. The margin of strength against failure was obviously not so great as in the case of specimen B, but the fact that an average shear stress in the vicinity of the shear yield strength of the material and about 40 percent greater than the theoretical buckling stress for the panels was developed seems indicative of fairly well-balanced proportions for shear-resistant web action. Accordingly, as far as the results of these few ultimate-strength tests are concerned, there appears to be no reason to question seriously the adequacy of the proposed stiffener formula for purposes of design.

In addition to the lateral deflections already discussed, figures 9 and 10 show the results of vertical-deflection measurements made at the center of the spans in the ultimate-load tests. Unfortunately, the elastic strength of the girders cannot be estimated from these data because small amounts of overstrain were produced unintentionally in some of the earlier tests. It is of interest to note, however, the close agreement obtained between measured and computed deflections within the elastic range indicated. In each case approximately two-thirds of the deflection was computed to be the result of shearing deformations, the remaining one-third was computed to be the result of flexure. Such girder proportions are not generally encountered in design, but apparently they present no difficulty as far as the estimation of probable deflections is concerned.

Figures 24 and 25 show the results of stress measurements on a number of the intermediate stiffeners of both girders. Although there is ample evidence of bending in the stiffeners, which deflected appreciably with the webs, no data were obtained to show that the stiffeners carried part of the shear by column action, as is the case for stiffeners on webs of the tension-field type. This observation is of interest in view of the requirement given in reference 5 (art. 226) that vertical stiffeners be designed as columns to resist a portion of the shear load, the amount depending upon the ratio of stiffener spacing to depth of web. According to the method of computation outlined in this specification, the intermediate stiffeners on the weaker half of specimen A, under a load of 80,000 pounds, should have been subjected to an average compressive stress of approximately 31,000 pounds per square inch. From the measurements shown in figure 24, such a stress condition was not produced. By the same requirement, the intermediate stiffeners on specimen B under a load of 160,000 pounds should have been subjected to an average compression of 15,000 pounds per square inch, which is also not supported by the stress measurements given.

The design of load-bearing stiffeners on the assumption of column action is perhaps a more logical procedure; although, as far as the results of these test are concerned, such a method appears quite conservative. Figures 26 and 27 show that the average measured stresses in the stiffeners near the top flange accounted for only about two-thirds of the applied load, while the stresses measured at the middle accounted for about one-third of the total. The ends of the load-bearing stiffeners on both girders were machined to fit closely between the fillets of the top and bottom flange angles. It should also be noted that the top of the web was flush with the face of the compression flange. This condition caused the web to be loaded directly in bearing on its extreme fibers rather than through the compression-flange rivets, as is usually the case.

The results of stress measurements on the top and bottom flanges of both girders are shown in figures 28 and 29. A very satisfactory agreement between average measured and computed bending stresses was obtained for the compression flange of specimen A, but in all other cases the measured values were considerably greater than those computed. Although it is not possible to account definitely for the discrepancies shown, the effect of gage length with respect to rivet spacing, the unequal distribution of load between the flange rivets, the effect of stress concentrations, and the lack of integral action are all possible contributing factors. Moments of inertia based upon net sections rather than gross sections would have provided a better agreement between measured and computed stresses in some cases, but there appears to be no logical reason for the use of net sections when an attempt is made to compute average stresses over gage lengths equal to the distance between rivet holes. From the good agreement between measured and computed vertical deflections previously shown in figures 9 and 10, it appears that these irregularities in measured stress were not reflected in the over-all behavior of the girders.

Table V gives the computed bending stresses corresponding to the maximum loads carried by both girders. It should be recognized that, since no evidence of flange failure other than plastic yielding was obtained, the values of stress given do not represent ultimate strengths. It may be pointed out, however, that the maximum stress computed for specimen B corresponds very closely to the theoretical buckling value, assuming that one edge of the flange is to be built in and the other edge is to be free. (See reference 9, tables 11 and 12.) The maximum computed stress for specimen A is about 20 percent less than the theoretical buckling value for the same edge conditions, an indication that a considerably higher value of flange stress might have been developed if failure in the web had not occurred.

The average bearing stresses on the flange rivets corresponding to the maximum applied loads were computed to be approximately 67,000 pounds per square inch in specimen A and 72,000 pounds per square inch in specimen B. After the ultimate-load tests had been completed, a portion of the top and bottom flange angles and the end load-bearing stiffeners were removed from the less severely damaged end of each girder for inspection of the rivet holes in the webs. From the measurements of hole distortion it appears that, even for the sides where the webs were still intact, the distribution of load between rivets ultimately obtained was not uniform. The largest changes in hole diameter, about 10 percent for specimen B, were in a direction consistent with the diagonal tension developed in the webs. The maximum changes in hole diameter found in specimen A, where a somewhat lower average shear stress was developed, were only about 2 percent. An examination of the rivet holes in the webs on the side where failures occurred was not made because of the severe local distortions produced and the uncertainty concerning the magnitude of the bearing stresses involved.

CONCLUSIONS

The results of this investigation are believed to justify the following conclusions:

1. Definite values for the flexural rigidity of stiffeners required to stiffen panels of given proportions, such as have been obtained by application of the buckling theory, apparently cannot be experimentally determined. Measurements of lateral deflection, as made in these tests, are useful in presenting a relative picture of web and stiffener behavior, but they do not permit a quantitative

determination of buckling resistance or stiffener effectiveness. Perhaps the most significant observation made, and the one that is also the most confusing from the standpoint of analysis, is that the buckling resistance of a web always may be increased by increasing the size of stiffener used until a condition of complete edge fixity is obtained for the subdivided panels.

2. The relative lateral deflections observed for the different sizes and types of stiffeners, whether of single- or double-angle type, were reasonably consistent with the computed stiffener moments of inertia. Effective widths of web equal to 25 percent of the clear depths were assumed for the single-angle stiffeners, although essentially the same results would have been obtained for most of the sizes considered if moments of inertia had been computed about the face of the web in contact with the stiffeners. This procedure is simpler from the standpoint of design but implies an appreciably different effective width of web for each size of angle, a condition that is not believed to be consistent with actual behavior. For large angles, moments of inertia computed about the face of the web may correspond to effective widths far greater than the stiffener spacing or the available web for each stiffener.

3. A comparison of the flexural rigidities obtainable from single- and double-angle stiffeners of similar proportions indicates the single-angle stiffeners to be more effective from the standpoint of stiffness-weight ratios.

4. The selection of stiffener proportions on the assumption that buckling will occur in the web for the load computed as critical for a condition of simply supported edges, as is done in the case of the stiffener theory, does not appear to be a conservative procedure as far as stiffener design is concerned in view of the appreciable edge restraint indicated for the web panels in many of the tests.

5. The following empirical formula is proposed as a tentative basis for the design of stiffeners on shear-resistant webs:

$$\lambda = \frac{14}{\left(\frac{b}{d}\right)^3}$$

where

λ ratio of flexural rigidity of one stiffener to flexural rigidity of web panel between adjacent stiffeners

b stiffener spacing, inch

d clear depth of web, inch

6. For most cases, apparently, the formula given in conclusion 5 provides stiffeners having more flexural rigidity than was indicated as necessary by either of the other two stiffener-design methods considered. Since there is no accepted basis for the determination of the requirements of an adequate stiffener for purposes of design, it is obviously difficult to evaluate different design methods. On the basis of the deflections observed in these tests, it hardly seems likely that the stiffener sizes proposed as adequate for the web panels investigated will be generally classed as too large. The stiffeners proposed are not, in general, so large as the stiffeners that would be required by current specifications for designs in structural steel.

7. As far as could be determined from ultimate-load tests on only two girders, each involving one size of stiffener on two different spacings, the proposed design method provides ample margin of strength against ultimate failure in the stiffeners. In both girders, the average shear stresses corresponding to the maximum applied loads were in the vicinity of the shear yield strength estimated for the web material. These maximum shear stresses also exceeded the theoretical buckling values for the weakest web panels by approximately 40 percent in specimen A and 90 percent in specimen B.

8. Although the strengths developed in the two girders were as high as would normally be considered obtainable in the design of shear-resistant webs of aluminum alloy 17S-T, it is significant that ultimate collapse and fracture did not occur until the connections between webs and stiffeners on the weaker half of the girders were broken. In specimen A, the full flexural rigidity of the stiffeners was apparently developed; in specimen B, the use of stronger stiffener connections would undoubtedly have increased the load-carrying capacity of the web.

9. The stress measurements made on a number of intermediate stiffeners on both girders provided no evidence

that these members should be designed as columns to resist a portion of the shear. The average measured stresses in the load-bearing stiffeners at the center of the spans, for sections within $2\frac{3}{8}$ inches of the top flange, accounted for only about two-thirds of the applied load. The stresses at the center of these stiffeners accounted for only one-third of the applied load.

10. The maximum computed bending stresses in the flanges for the loads producing web failures were 28,100 pounds per square inch in specimen A and 33,700 pounds per square inch in specimen B. The value for specimen B corresponds closely to the theoretical buckling stress for the outstanding flange, assuming one edge built in and the other edge free. No evidence of primary flange failure was obtained.

11. Within the apparent elastic range, the measured vertical deflections at the center of the spans were in very close agreement with the computed values. Approximately two-thirds of these deflections were computed to be the result of shear; the remaining one-third were computed to be the result of flexure.

12. The average computed bearing stresses between flange rivets and webs for the maximum applied loads were approximately 67,000 pounds per square inch in specimen A and 72,000 pounds per square inch in specimen B. An examination of some of the rivet holes in the webs for the half of the girders still intact indicated permanent distortions in the direction of the diagonal-tensile stresses ultimately developed. The maximum increases in hole diameter were about 2 percent in specimen A and 10 percent in specimen B.

RECOMMENDATIONS

It is recognized that the proof of the dependability of any proposed new method of stiffener design requires more experimental verification than was obtained in this investigation. It is proposed, therefore, as an essential step in the formulation of a satisfactory solution to the stiffener problem, that an additional series of aluminum-alloy 17S-T plate girders be fabricated for test purposes. The principal object of these new tests should be to compare the method of stiffener design proposed in this report with other methods on girders representing more balanced proportions of conventional design. The intermediate stiffeners should extend the full depth between flanges rather than over only the clear depth of web, and they should be riveted rather than bolted to the webs. Each girder should involve only one size and spacing of intermediate stiffeners and should be used for only one test, and that test should be carried to failure. Such an investigation not only would provide comparative data on methods of proportioning intermediate stiffeners but also would make possible some analysis of the present design methods of providing a reasonable equality in shear and flexural strengths.

Aluminum Research Laboratories,
Aluminum Company of America,
New Kensington, Penna., December 19, 1941.

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TABLE I.– PROPERTIES OF 17S–T PLATE-GIRDER MATERIAL

Girder	Material[a]	Yield strengths (offset = 0.2 percent)		Tensile strength	Elongation in 2 inches
		Tension[b] (lb/sq in.)	Compression[c] (lb/sq in.)	(lb/sq in.)	(percent)
Specimen A	Web–X	44,800	47,300	63,900	18.0
	Web–W	51,100	40,900	65,800	22.5
	Flange–W	48,700	42,800	68,800	18.0
Specimen B	Web–X	42,800	44,400	65,600	20.5
	Web–W	48,300	39,400	66,200	21.0
	Flange–W	49,300	44,300	71,500	19.0

[a]Web material 1/8 in. thick.
Flanges: 3- by 2- by 5/16–in. extruded angles, Specimen A
4- by 3- by 3/8–in. extruded angles, Specimen B
X indicates cross-grain specimen.
W indicates with-grain specimen.

[b]Tests made on standard rectangular tension specimens with 2–in. gage lengths. (See fig. 2 of reference 6.)

[c]Tests on web made on 0.125- by 5/8- by $2\frac{5}{8}$–in. specimens by single-thickness method. (See reference 7.) Tests on flanges made on 5/8- by 2–in. specimens of full thickness, tested as columns with flat ends.

TABLE II.- SUMMARY OF MEASURED LATERAL DEFLECTIONS AND PERMANENT SETS FOR WEBS AND STIFFENERS

Number and size of panels	Number of stiffeners	Number and size of angles per stiffener	Moment of inertia per stiffener ($in.^4$) (a)	Theoretical buckling load per panel (lb) (b)	Corresponding average shear stress (lb/sq in.)	Test load (lb) (c)	Maximum web deflections midway between stiffeners (0.001 in.) Range	Maximum web deflections midway between stiffeners (0.001 in.) Average all panels	Maximum stiffener deflections (0.001 in.) Range	Maximum stiffener deflections (0.001 in.) Average all stiffeners	Maximum permanent sets (0.001 in.) Web	Maximum permanent sets (0.001 in.) Stiffeners
				SPECIMEN A								
1 - 12x24 in.	None	———	———	26,300	6,570	35,000	13	———	———	———	—	———
						45,000	133	———	———	———	5	———
2 - 12x12 in. (Series I)	1	1 - 1/2x1/2x1/16	0.0040	38,700	9,690	40,000	14-15	15	———	9	—	———
						50,000	86-111	98	———	117	8	13
		2 - 1/2x1/2x1/16	.0076	38,700	9,690	40,000	14-30	17	———	15	—	———
						50,000	66-76	71	———	81	7	6
		1 - 3/4x3/4x3/32	.0136	38,700	9,690	40,000	9-14	12	———	4	—	———
						55,000	108-113	110	———	56	4	2
		2 - 3/4x3/4x3/32	.0306	38,700	9,690	40,000	13-17	15	———	5	—	———
						60,000	125-139	132	———	33	6	4
		1 - 3/4x3/4x3/16	.0264	38,700	9,690	40,000	29-30	30	———	5	—	———
						60,000	128-132	130	———	38	3	2
		2 - 3/4x3/4x3/16	.0706	38,700	9,690	40,000	20-24	22	———	4	—	———
						65,000	147-174	161	———	20	11	3
3 - 8x12 in. (Series II)	2	1 - 3/4x3/4x3/32	0.0136	66,400	16,600	40,000	16-37	26	24-25	25	—	———
						55,000	42-80	63	65-69	67	6	5
		2 - 3/4x3/4x3/32	.0306	66,400	16,600	50,000	12-49	29	22-23	23	—	———
						70,000	37-111	76	66-75	70	7	9
		1 - 3/4x3/4x3/16[d]	.0264	66,400	16,600	60,000	6-21	13	3-6	4	—	———
						70,000	34-109	80	47-51	49	15	8
		2 - 3/4x3/4x3/16	.0706	66,400	16,600	60,000	7-61	38	14-21	17	—	———
						70,000	24-89	52	27-29	28	2	1
		1 - 1x3/4x1/8	.0400	66,400	16,600	60,000	18-75	45	21-23	22	—	———
						70,000	36-107	71	33-48	40	8	3
		2 - 1x3/4x1/8	.101	66,400	16,600	60,000	11-65	34	9-12	11	—	———
						70,000	28-84	51	15-19	17	6	2
4 - 6x12 in. (Series III)	3	1 - 3/4x3/4x3/32	0.0136	105,200	26,300	60,000	7-25	16	3-24	15	—	———
						70,000	22-82	52	36-80	52	7	7
		2 - 3/4x3/4x3/32	.0306	105,200	26,300	60,000	6-18	12	1-14	7	—	———
						75,000	14-48	28	10-37	20	10	2
		1 - 3/4x3/4x3/16	.0264	105,200	26,300	60,000	3-5	4	3-5	4	—	———
						75,000	10-17	13	4-15	11	3	1

[a]Moments of inertia for single-angle stiffeners include effective width of web equal to 25 percent of clear depth between flanges. Web neglected in cases of double-angle stiffeners. See table III.

[b]Based on assumption of simply supported edges. See table 17 of reference 9.

[c]Larger test load was maximum applied in each case. Smaller test load selected to show, by comparison with larger load, change in rate of deflection.

[d]First test made in this series. Permanent sets produced resulted in relatively large deflections for subsequent tests.

[e]Check tests made after series V had been completed.

TABLE II.- SUMMARY OF MEASURED LATERAL DEFLECTIONS AND PERMANENT SETS FOR WEBS AND STIFFENERS

Number and size of panels	Number of stiffeners	Number and size of angles per stiffener	Moment of inertia per stiffener (in.^{4}) (a)	Theoretical buckling load per panel (lb) (b)	Corresponding average shear stress (lb/sq in.)	Test load (lb) (c)	Maximum web deflections midway between stiffeners (0.001 in.)		Maximum stiffener deflections (0.001 in.)		Maximum permanent sets (0.001 in.)	
							Range	Average all panels	Range	Average all stiffeners	Web	Stiffeners
				SPECIMEN B								
1 - 24x48 in.	None	—	—	12,300	1,640	16,000	25	—	—	—	—	—
						30,000	239	—	—	—	5	—
2 - 24x24 in. (Series I)	1	1 - 1/2x1/2x1/16	0.0046	18,100	2,420	20,000	2-27	14	—	16	—	—
						28,000	111-179	145	—	83	1	3
		2 - 1/2x1/2x1/16	.0076	18,100	2,420	20,000	10-18	14	—	21	—	—
						30,000	115-133	124	—	132	2	3
		1 - 3/4x3/4x3/32	.0150	18,100	2,420	20,000	5-13	9	—	9	—	—
						30,000	64-122	93	—	113	4	3
		2 - 3/4x3/4x3/32	.0306	18,100	2,420	20,000	15-16	16	—	8	—	—
						30,000	68-131	100	—	50	5	2
		2 - 3/4x3/4x3/32[c]	.0306	18,100	2,420	20,000	30-32	31	—	7	—	—
						30,000	119-124	122	—	43	3	3
		1 - 3/4x3/4x3/16	.0304	18,100	2,420	20,000	8-18	14	—	3	—	—
						30,000	97-113	115	—	43	3	2
		2 - 3/4x3/4x3/16	.0706	18,100	2,420	20,000	10-12	11	—	3	—	—
						36,000	155-167	161	—	17	4	3
		1 - 1x3/4x1/8	.0447	18,100	2,420	20,000	20-22	21	—	3	—	—
						30,000	108-128	118	—	36	2	1
		2 - 1x3/4x1/8	.101	18,100	2,420	20,000	11-16	13	—	2	—	—
						36,000	150-158	154	—	16	2	2
		1 - 1-1/4x1x1/8	.0826	18,100	2,420	20,000	14-20	17	—	1	—	—
						36,000	165-167	166	—	30	1	2
		2 - 1-1/4x1x1/8	.192	18,100	2,420	24,000	5-10	8	—	1	—	—
						36,000	149-152	150	—	12	1	1
		2 - 1-1/4x1x1/8[c]	.192	18,100	2,420	20,000	23-33	28	—	2	—	—
						34,000	148-155	152	—	7	4	3
3 - 16x24 in. (Series II)	2	1 - 3/4x3/4x3/16	0.0304	31,100	4,150	25,000	10-35	22	0-7	4	—	—
						35,000	40-103	70	27-39	33	3	4
		2 - 3/4x3/4x3/16	.0706	31,100	4,150	30,000	17-24	20	3-9	6	—	—
						45,000	105-147	122	38-66	52	2	3
		1 - 1x3/4x1/8	.0447	31,100	4,150	30,000	9-14	11	1-9	5	—	—
						40,000	65-112	85	56-68	62	2	2
		2 - 1x3/4x1/8	.101	31,100	4,150	35,000	13-19	16	10-12	11	—	—
						55,000	118-147	134	39-64	52	2	2
		1 - 1-1/4x1x1/8	.0828	31,100	4,150	35,000	18-40	28	2-15	8	—	—
						50,000	104-159	136	28-64	63	3	1
		2 - 1-1/4x1x1/8	.192	31,100	4,150	35,000	6-25	14	5-7	6	—	—
						65,000	168-185	177	24-48	36	3	1
		1 - 1-1/2x1x5/32	.168	31,100	4,150	35,000	8-45	31	2-8	6	—	—
						55,000	139-185	166	36-43	40	7	4
		2 - 1-1/2x1x5/32	.420	31,100	4,150	35,000	20-27	24	5-6	6	—	—
						65,000	151-182	165	20-30	25	2	4

TABLE II.- SUMMARY OF MEASURED LATERAL DEFLECTIONS AND PERMANENT SETS FOR WEBS AND STIFFENERS

Number and size of panels	Number of stiffeners	Number and size of angles per stiffener	Moment of inertia per stiffener (in.^{4}) (a)	Theoretical buckling load per panel (lb) (b)	Corresponding average shear stress (lb/sq in.)	Test load (lb) (c)	Maximum web deflections midway between stiffeners (0.001 in.) Range	Maximum web deflections midway between stiffeners (0.001 in.) Average all panels	Maximum stiffener deflections (0.001 in.) Range	Maximum stiffener deflections (0.001 in.) Average all stiffeners	Maximum permanent sets (0.001 in.) Web	Maximum permanent sets (0.001 in.) Stiffeners
Specimen B (Continued)												
4 - 12x24 in. (Series III)	3	1 - 1x3/4x1/8	0.0447	49,200	6,570	35,000	10-28	19	6-17	13	---	---
						45,000	42-74	61	42-61	54	4	4
		2 - 1x3/4x1/8	.101	49,200	6,570	45,000	3-34	12	1-14	6	---	---
						55,000	55-99	81	36-53	46	6	4
		1 - 1-1/4x1x1/8	.0886	49,200	6,570	40,000	23-51	32	11-24	17	---	---
						50,000	54-99	75	40-57	49	3	4
		2 - 1-1/4x1x1/8	.192	49,200	6,570	50,000	12-38	22	3-6	4	---	---
						70,000	91-123	111	32-56	47	4	6
		1 - 1-1/2x1x5/32	.168	49,200	6,570	45,000	18-48	30	6-17	10	---	---
						60,000	83-117	96	36-44	40	3	4
		1 - 1-1/2x1x5/32	.420	49,200	6,570	50,000	1-36	9	2-9	5	---	---
						80,000	85-163	134	32-49	40	4	3
		1 - 1-1/2x1x1/4	.231	49,200	6,570	50,000	12-46	29	3-13	5	---	---
						70,000	112-130	119	39-47	44	4	6
		2 - 1-1/2x1x1/4	.632	49,200	6,570	60,000	6-71	39	4-6	5	---	---
						90,000	108-212	159	19-29	22	7	3
		1 - 1-3/4x1-1/4x1/4	.366	49,200	6,570	60,000	58-93	78	3-14	11	---	---
						80,000	129-165	147	16-42	31	5	5
		2 - 1-3/4x1-1/4x1/4	1.03	49,200	6,570	60,000	9-87	48	4-9	6	---	---
						90,000	81-204	148	12-21	17	2	3
6 - 8x24 in. (Series IV)	5	1 - 1-1/4x1x1/8	0.0826	101,200	13,500	60,000	7-26	14	2-17	10	---	---
						80,000	39-93	64	33-59	47	2	7
		2 - 1-1/4x1x1/8	.0192	101,200	13,500	80,000	8-78	31	9-37	16	---	---
						100,000	31-124	78	16-69	44	3	3
		1 - 1-1/2x1x5/32	.168	101,200	13,500	70,000	4-38	20	3-18	12	---	---
						90,000	29-72	59	13-37	28	4	5
		2 - 1-1/2x1x5/32	.420	101,200	13,500	80,000	5-52	23	7-19	11	---	---
						110,000	18-122	75	12-42	25	11	4
		1 - 1-1/2x1x1/4	.231	101,200	13,500	80,000	5-43	20	3-13	7	---	---
						110,000	44-119	70	16-48	34	5	6
		2 - 1-1/2x1x1/4	.632	101,200	13,500	90,000	6-65	22	7-15	11	---	---
						130,000	40-145	96	12-46	23	23	3
		1 - 1-3/4x1-1/4x1/4	.366	101,200	13,500	90,000	9-78	39	8-19	11	---	---
						120,000	40-128	81	14-45	25	4	5
		2 - 1-3/4x1-1/4x1/4	1.03	101,200	13,500	90,000	5-77	33	6-14	10	---	---
						130,000	39-118	73	10-28	17	8	3
8 - 6x24 in. (Series V)	7	1 - 1-1/2x1x5/32	0.168	174,000	23,200	80,000	12-37	22	1-15	10	---	---
						110,000	37-87	56	18-53	34	3	3
		2 - 1-1/2x1x5/32	.420	174,000	23,200	100,000	6-48	18	4-18	8	---	---
						130,000	12-107	51	7-40	18	4	8
		1 - 1-1/2x1x1/4	.231	174,000	23,200	100,000	3-63	22	1-18	8	---	---
						140,000	39-167	82	10-66	60	18	9
		2 - 1-1/2x1x1/4	.632	174,000	23,200	120,000	12-81	28	9-28	16	---	---
						150,000	16-127	49	15-44	23	9	6
		1 - 1-3/4x1-1/4x1/4	.366	174,000	23,200	120,000	6-103	39	6-35	17	---	---
						150,000	32-153	72	13-59	36	9	5

TABLE III.- ELEMENTS OF INTERMEDIATE STIFFENERS

Size of stiffener angle (in.)	Area of one angle (sq in.)	Stiffener on one side of web only			I for stiffeners on both sides of web[a] (in.^{4})
		I for effective width of web equal to 25 percent of clear depth (in.^{4})		I for angle about face of web (in.^{4})	
		Specimen A	Specimen B	Specimens A and B	Specimens A and B
1/2 x 1/2 x 1/16	0.059	0.0040	0.0046	0.0026	0.0076
3/4 x 3/4 x 3/32	.132	.0136	.0150	.0116	.0306
3/4 x 3/4 x 3/16	.246	.0264	.0304	.0267	.0706
1 x 3/4 x 1/8	.202	.0400	.0447	.0413	.101
$1\frac{1}{4}$ x 1 x 1/8	.27	.0737	.0826	.0820	.192
$1\frac{1}{2}$ x 1 x 5/32	.37	.147	.168	.185	.420
$1\frac{1}{2}$ x 1 x 1/4	.56	.198	.231	.276	.632
$1\frac{3}{4}$ x $1\frac{1}{4}$ x 1/4	.69	.314	.366	.462	1.03

[a]Effective width of web neglected.

TABLE IV.- COMPARISON OF STIFFENER SIZES COMPUTED BY DIFFERENT DESIGN METHODS

[Example of plate-girder design (120- x 5/16-inch web; 112.5-inch clear depth) from reference 5, art. 805]

Stiffener spacing (in.)	Number of stiffeners	Required moment of inertia, I, per stiffener ($in.^4$)			Required size for single bulb-angle stiffeners[a]			Required size for ordinary double-angle stiffeners[b]			
		Reference 3	Reference 1[c]	Proposed	Reference 3	Reference 1	Proposed	Reference 3	Reference 1	Proposed	Steel specifications (reference 12)
32.6	8	22.46	-------	53.5	5x3x5/16 A = 2.99 I_{3-3}=24.9 I_{4-4}=22.7	----------	6-1/2x3x3/8 A = 4.27 I_{3-3}=63.6 I_{4-4}=53.3	5x3x5/16 A = 4.80 I_{1-1}=28.7	----------	6x3-1/2x3/8 A = 6.86 I_{1-1}= 57.1	6x3-1/2x3/8 A = 6.86 I_{1-1}=57.1
41.5	6	22.46	19.5	33.1	5x3x5/16 A = 2.99 I_{3-3}=24.9 I_{4-4}=22.7	5x3x5/16 A = 2.99 I_{3-3}=24.9 I_{4-4}=22.7	6x3x5/16 A = 3.31 I_{3-3}=40.1 I_{4-4}=35.5	5x3x5/16 A = 4.80 I_{1-1}=28.7	5x3x5/16 A = 4.80 I_{1-1}=28.7	5x3x3/8 A = 5.72 I_{1-1}=34.4	6x3-1/2x3/8 A = 6.86 I_{1-1} = 57.1
61.2	4	8.62	7.7	15.2	4x3x1/4 A = 2.07 I_{3-3}=10.2 I_{4-4}=10.1	4x3x1/4 A = 2.07 I_{3-3}=10.2 I_{4-4}=10.1	5x3x5/16 A = 2.99 I_{3-3}=24.9 I_{4-4}=22.7	4x3x1/4 A = 3.38 I_{1-1}=12.2	3-1/2x3x1/4 A = 3.12 I_{1-1}=8.3	4x3x5/16 A = 4.18 I_{1-1}=15.2	6x3-1/2x3/8 A = 6.86 I_{1-1}=57.1

[a]Selected from table 21 of reference 5 for aluminum alloy 27S-T structures. A = area of stiffener, sq in.; I_{3-3} = moment of inertia of angle alone, about face of web, $in.^4$; I_{4-4} = moment of inertia for angle plus effective width of web equal to 25 percent of clear depth, $in.^4$.

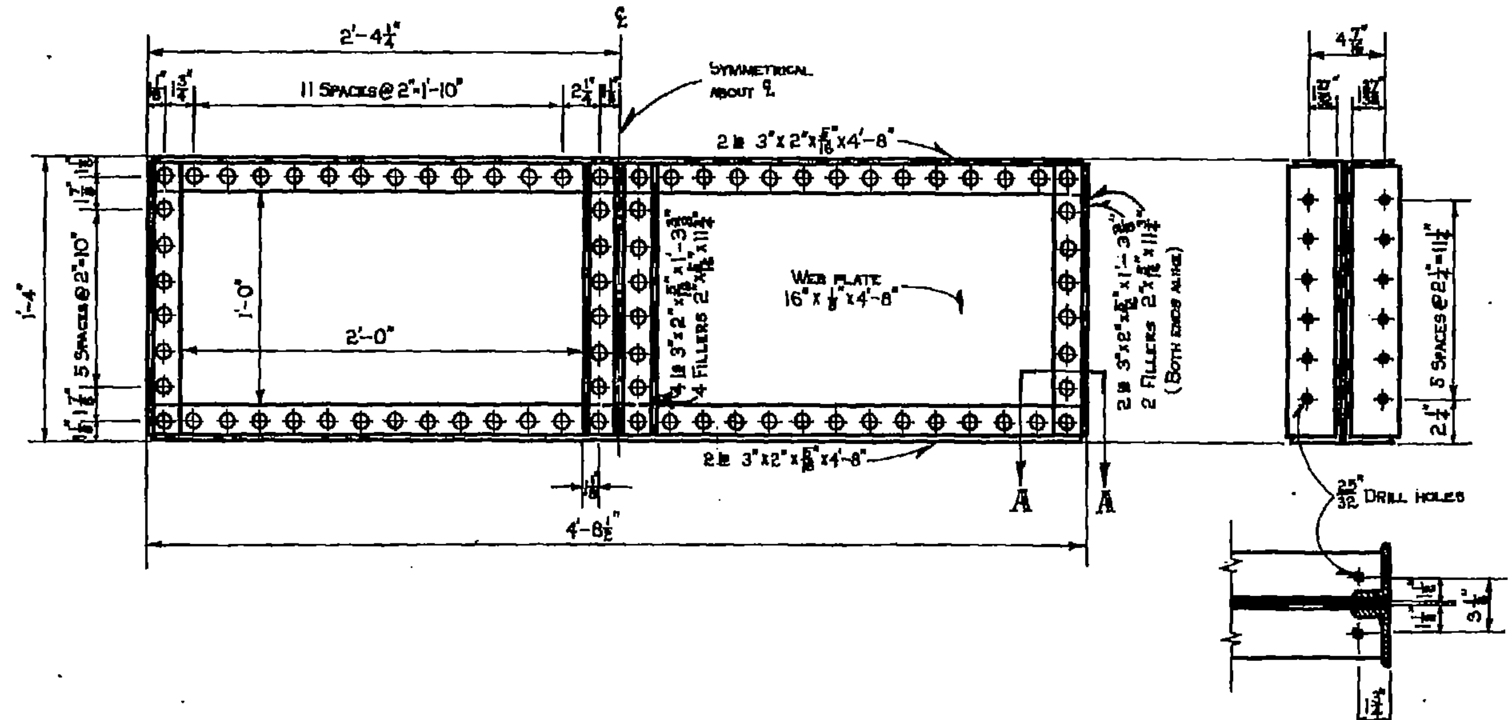
[b]I_{1-1} = moment of inertia about center line of web.

[c]Values of I are 100 percent greater than theoretical values for case of one stiffener. See reference 1, p. 417.

TABLE V.- ULTIMATE STRENGTHS OF PLATE GIRDERS UNDER CENTRAL CONCENTRATED LOADS

[See figs. 9 and 10 for sizes and spacings of intermediate stiffeners in ultimate-load tests]

Specimen	Over-all depth (in.)	Span (ft)	Span (in.)	Ultimate load (lb)	Corresponding average shear stress (lb/sq in.)	Corresponding maximum bending stress (lb/sq in.)	Remarks
A	16	4	$8\frac{1}{2}$	93,300	23,300	28,100	Web collapsed and fractured after all stiffener connections on weaker half of girder were broken.
B	30	9	1/2	191,500	25,500	33,700	Web collapsed and fractured after connections for two end stiffeners on weaker half were broken. End rivet in compression flange also sheared off.

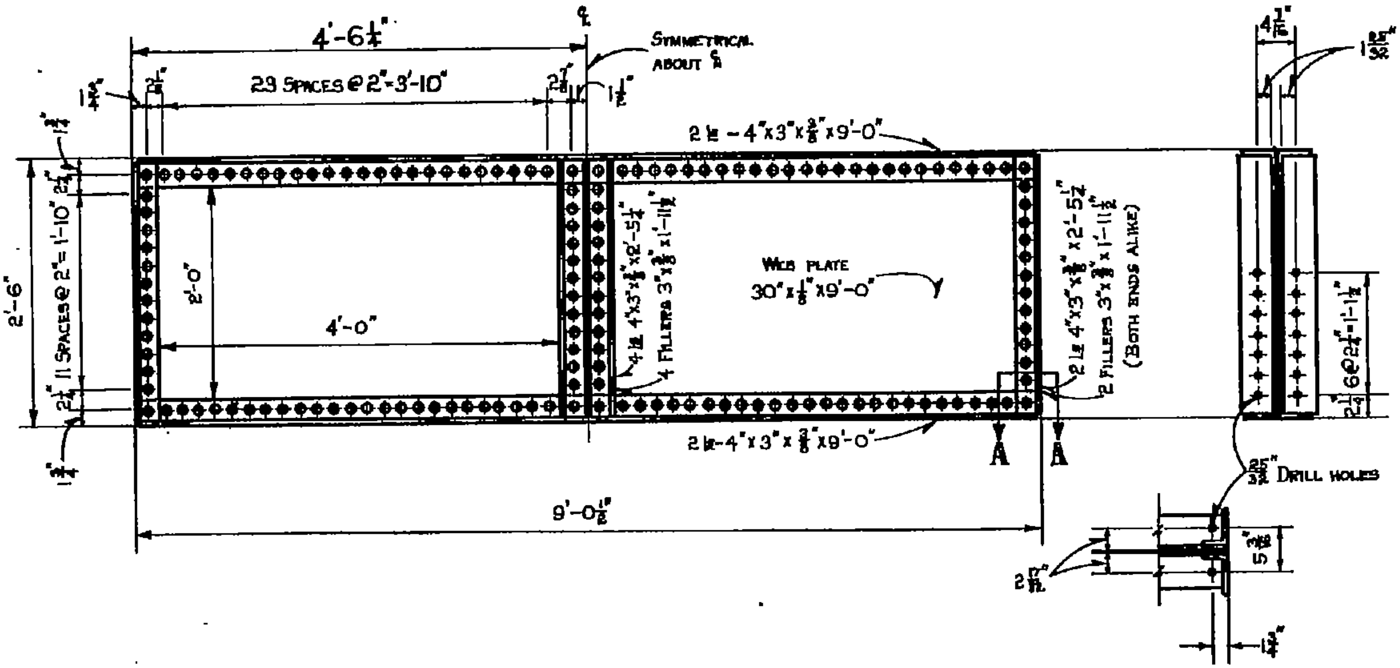


NOTE: (1) ALL MATERIAL ALUMINUM ALLOY 17S-T
(2) ALL RIVETS ¾" DIAM. DRIVEN HOT WITH CONE-POINT HEADS IN 25/32" DRILLED HOLES
(3) STIFFENERS HAVE CLOSE FIT BETWEEN FLANGE ANGLES

COMPUTED STRESS AND DEFLECTION FACTORS FOR CENTRAL CONCENTRATED LOAD P, BASED ON ELEMENTS OF GROSS SECTION:

AVERAGE SHEAR STRESS = 0.250 P.
MAXIMUM SHEAR STRESS = 0.278 P.
MAXIMUM BENDING STRESS = 0.301 P.
MAXIMUM VERTICAL DEFLECTION = 0.00000880 P. {35% FLEXURE, 65% SHEAR}

FIG. 1.- PLATE GIRDER FOR STIFFENER TESTS - SPECIMEN A.

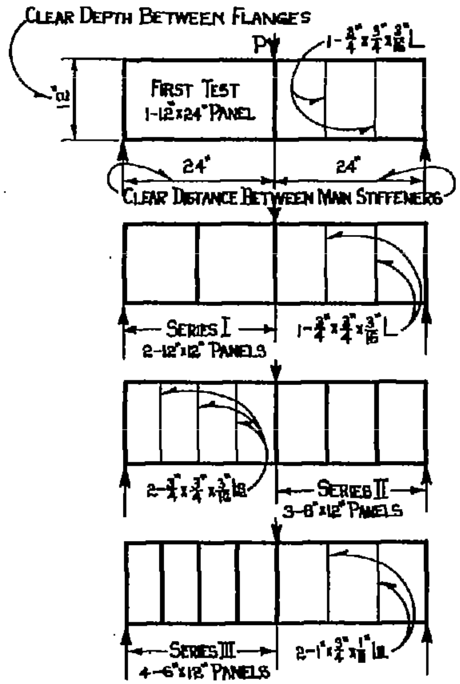


NOTE: (1) ALL MATERIAL ALUMINUM ALLOY 17S-T
(2) ALL RIVETS ⅝" DIAM. DRIVEN HOT WITH CONE-POINT HEADS IN 21/32" DRILLED HOLES
(3) STIFFENERS HAVE CLOSE FIT BETWEEN FLANGE ANGLES.

COMPUTED STRESS AND DEFLECTION FACTORS FOR CENTRAL CONCENTRATED LOAD P, BASED ON ELEMENTS OF GROSS SECTION:

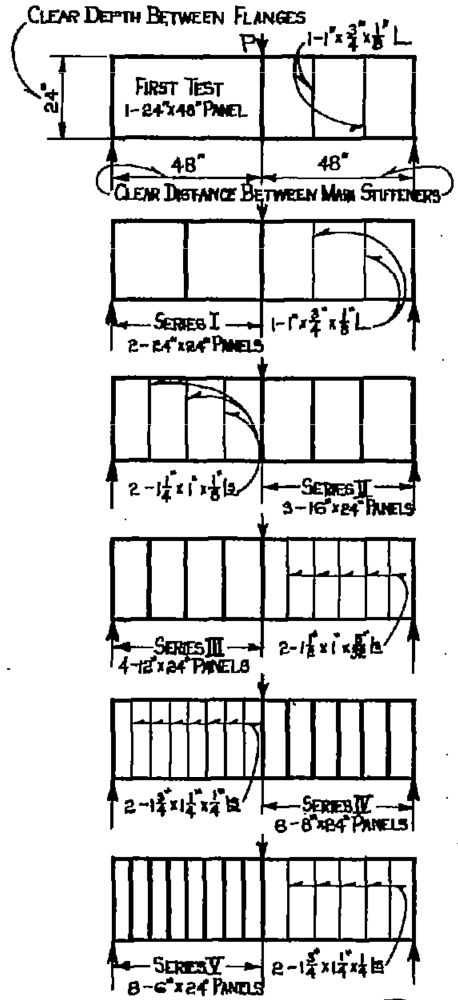
AVERAGE SHEAR STRESS = 0.133 P
MAXIMUM SHEAR STRESS = 0.147 P
MAXIMUM BENDING STRESS = 0.176 P
MAXIMUM VERTICAL DEFLECTION = 0.0000289 P { 37% FLEXURE, 63% SHEAR

FIG. 2.- PLATE GIRDER FOR STIFFENER TESTS - SPECIMEN B.



Shear Buckling Load P for Test Panels* Lb.	Corresponding Average Shear Stress psi
26,300	6,570
38,700	9,690
66,400	16,600
105,200	26,300

Specimen A



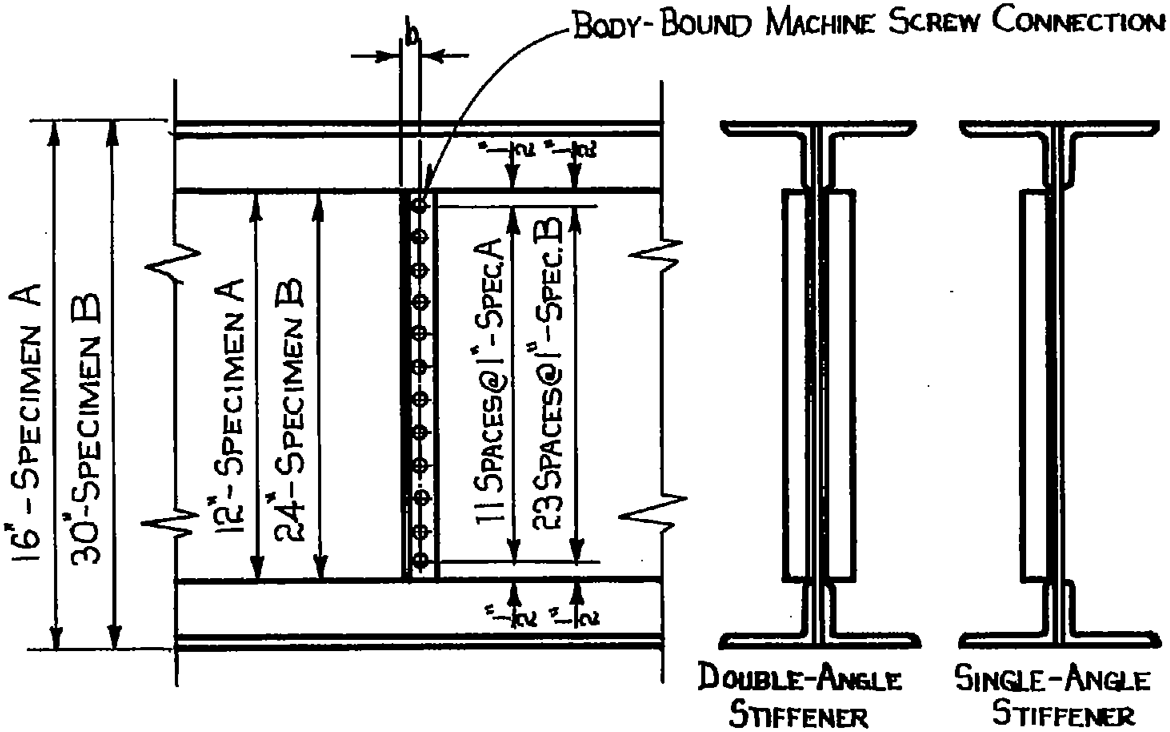
Shear Buckling Load P for Test Panels* Lb.	Corresponding Average Shear Stress psi
12,300	1,640
18,100	2,420
31,100	4,150
49,200	6,570
101,200	13,500
174,000	23,200

Specimen B

* Theoretical Value for Panels With Simply Supported Edges

Fig. 3.- Schedule of Stiffener Tests

See Table II for Sizes In Each Series



SIZE OF STIFFENER ANGLE	WIDTH OF LEG ATTACHED TO WEB	GAGE DISTANCE b
$\frac{1}{2}" \times \frac{1}{2}" \times \frac{1}{16}"$	$\frac{1}{2}"$	$\frac{5}{16}"$
$\frac{3}{4}" \times \frac{3}{4}" \times \frac{3}{32}"$	$\frac{3}{4}"$	$\frac{7}{16}"$
$\frac{3}{4}" \times \frac{3}{4}" \times \frac{3}{16}"$	$\frac{3}{4}"$	$\frac{7}{16}"$
$1" \times \frac{3}{4}" \times \frac{1}{8}"$	$\frac{3}{4}"$	$\frac{7}{16}"$
$1\frac{1}{4}" \times 1" \times \frac{1}{8}"$	$1"$	$\frac{1}{2}"$
$1\frac{1}{2}" \times 1" \times \frac{5}{32}"$	$1"$	$\frac{1}{2}"$
$1\frac{1}{2}" \times 1" \times \frac{1}{4}"$	$1"$	$\frac{5}{8}"$
$1\frac{3}{4}" \times 1\frac{1}{4}" \times \frac{1}{4}"$	$1\frac{1}{4}"$	$\frac{5}{8}"$

No. 6-32 N.C. (0.138" DIA.) ALUMINUM ALLOY 17 S-T SCREWS FOR $\frac{1}{2}" \times \frac{1}{2}" \times \frac{1}{16}"$ Ls
No. 10-32 N.F. (0.190" DIA.) ALUMINUM ALLOY 17 S-T SCREWS FOR ALL OTHER SIZES
ALL HOLES DRILLED & REAMED FROM TEMPLATES TO BODY SIZE OF SCREWS

FIG. 4.- INTERMEDIATE STIFFENERS FOR PLATE GIRDER SPECIMENS A & B

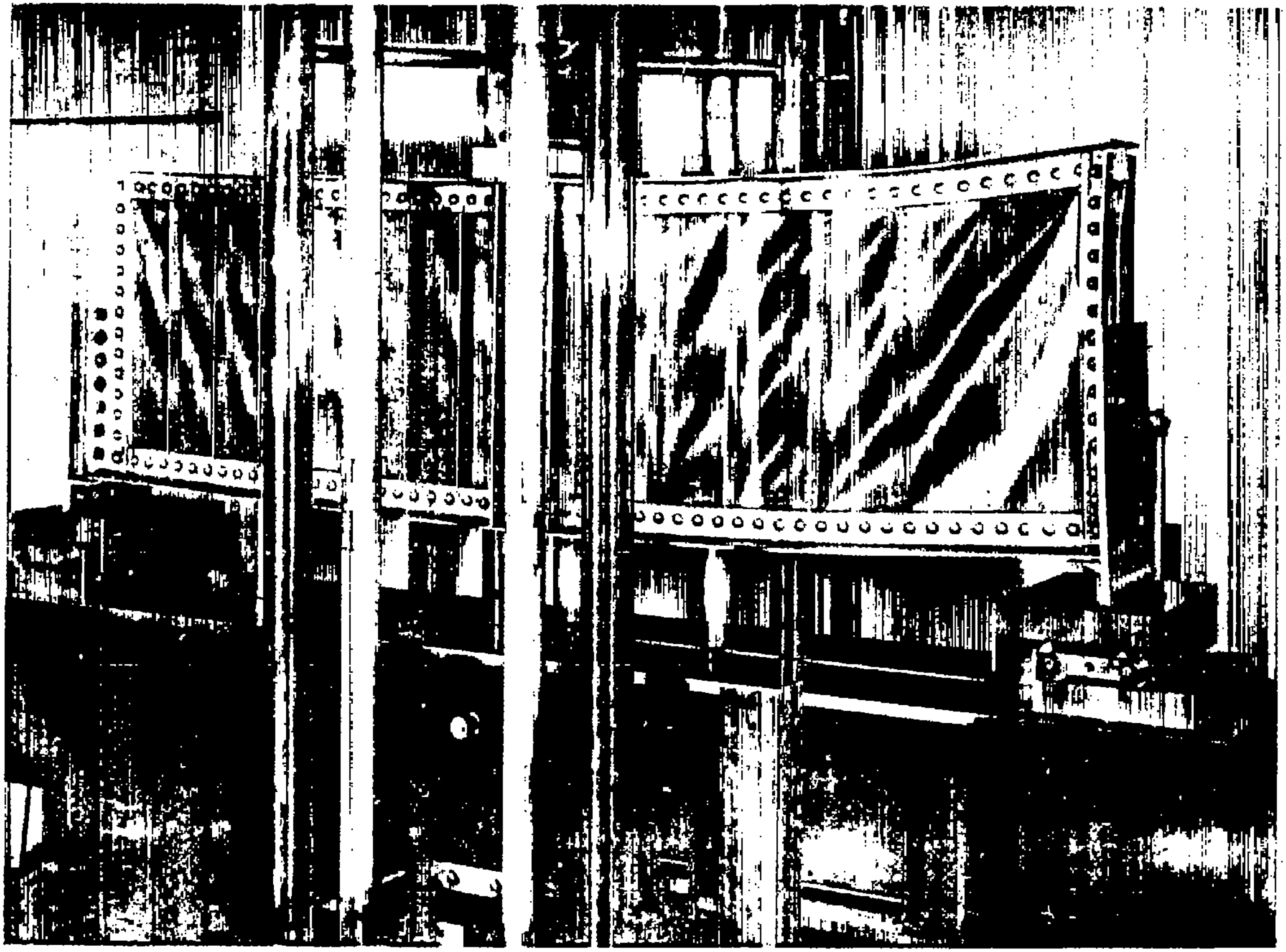


Figure 5.- Girder tests in 300,000-pound-capacity Amsler machine (specimen B after failure).

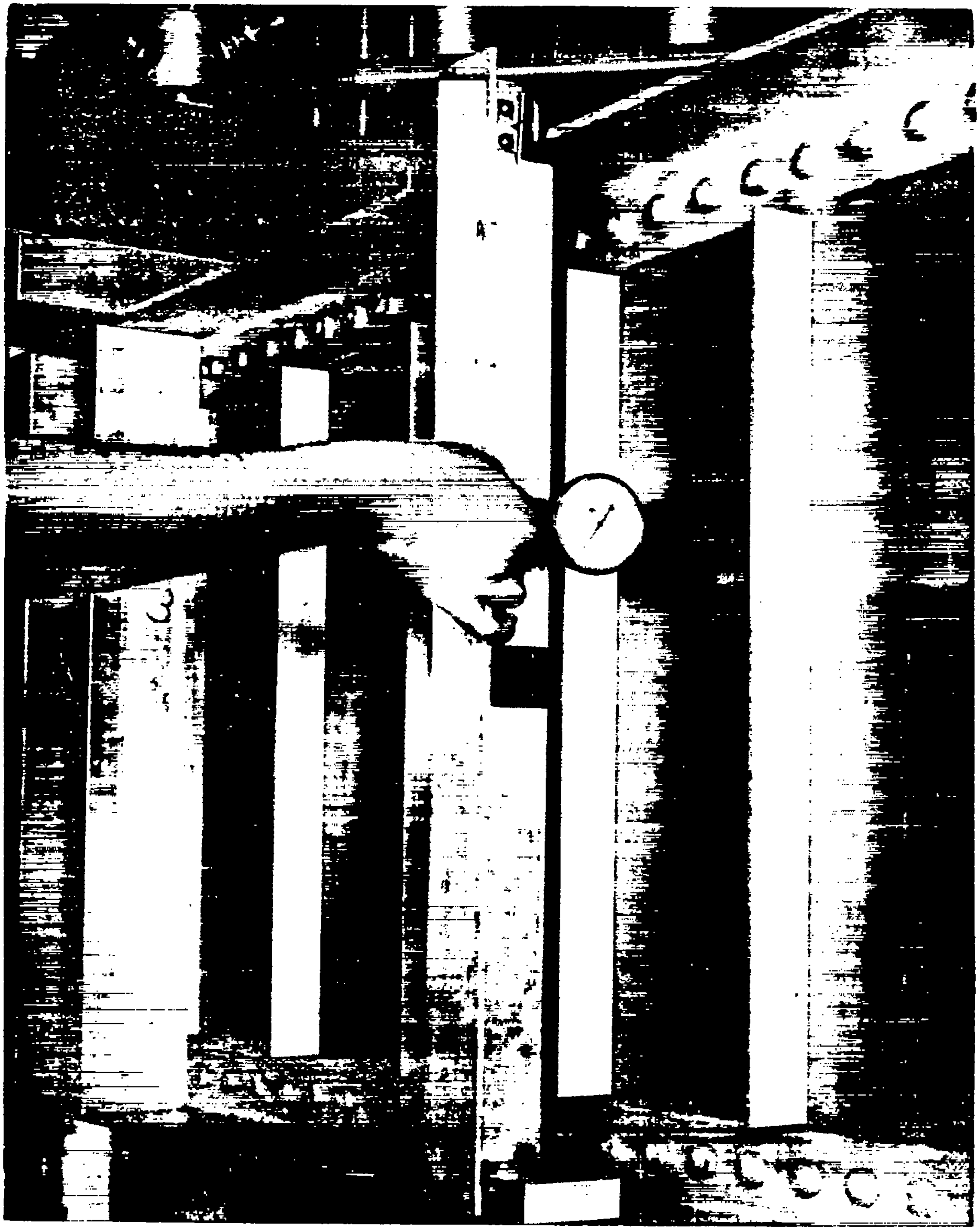


Figure 6.- Apparatus for measurement of lateral deflections (reference angle held against flanges by tension springs hooked over opposite edges).

1 - 12 x 24 in. panel. Thickness - 1/8 in.
2 - 3 x 2 x 5/16 in. angles on outside edges.

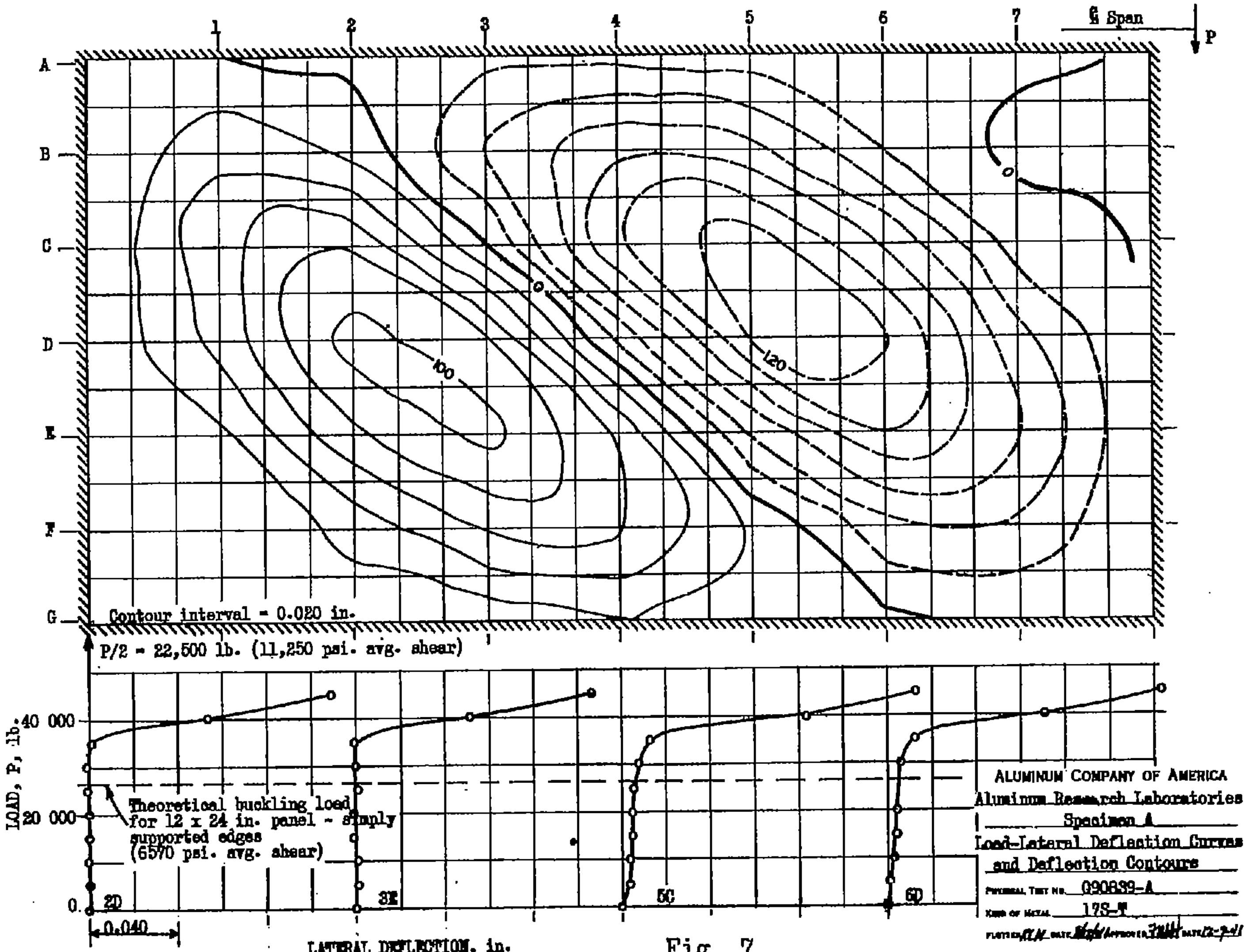


Fig. 7

2 - 12 x 12 in. panels. Thickness = 1/8 in.
2 - 3 x 2 x 5/16 in. angles on outside edges.
2 - 1/2 x 1/2 x 1/16 in. angles at Section 4. I = 0.0076 in.^{4}
(I proposed from tests = 0.0307 in.^{4})
(I proposed by Moisseiff and Lienhard = 0.0141 in.^{4})

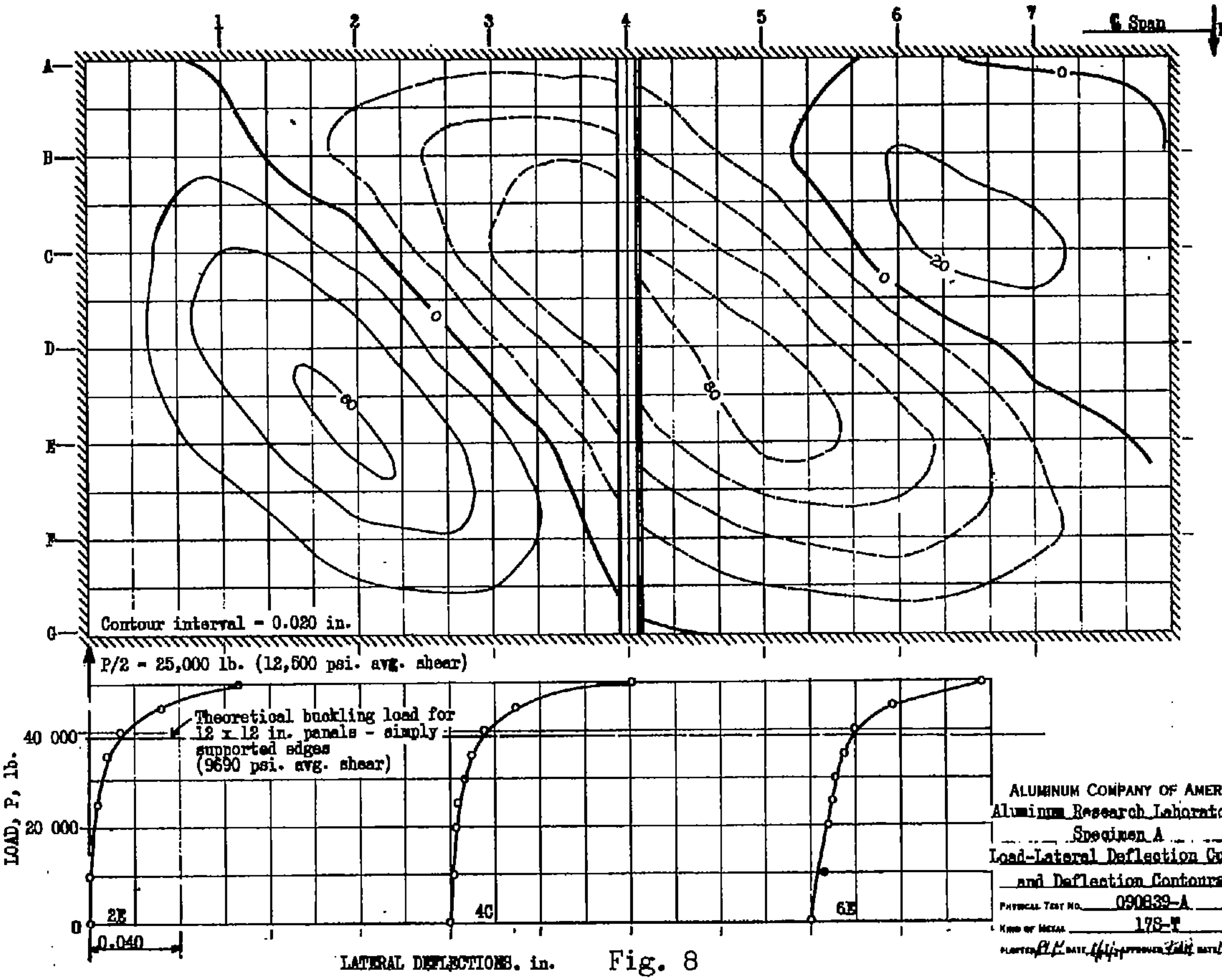


Fig. 8

ALUMINUM COMPANY OF AMERICA
Aluminum Research Laboratories
Specimen A
Load-Lateral Deflection Curves
and Deflection Contours
Physical Test No. 090839-A
Kind of Metal 17S-T

$A_v = 2 - .202 = .404$
$dt = 8 \times \frac{1}{8} = 1.00$
$A_v/dt = .404$
$h/t \sim 100$

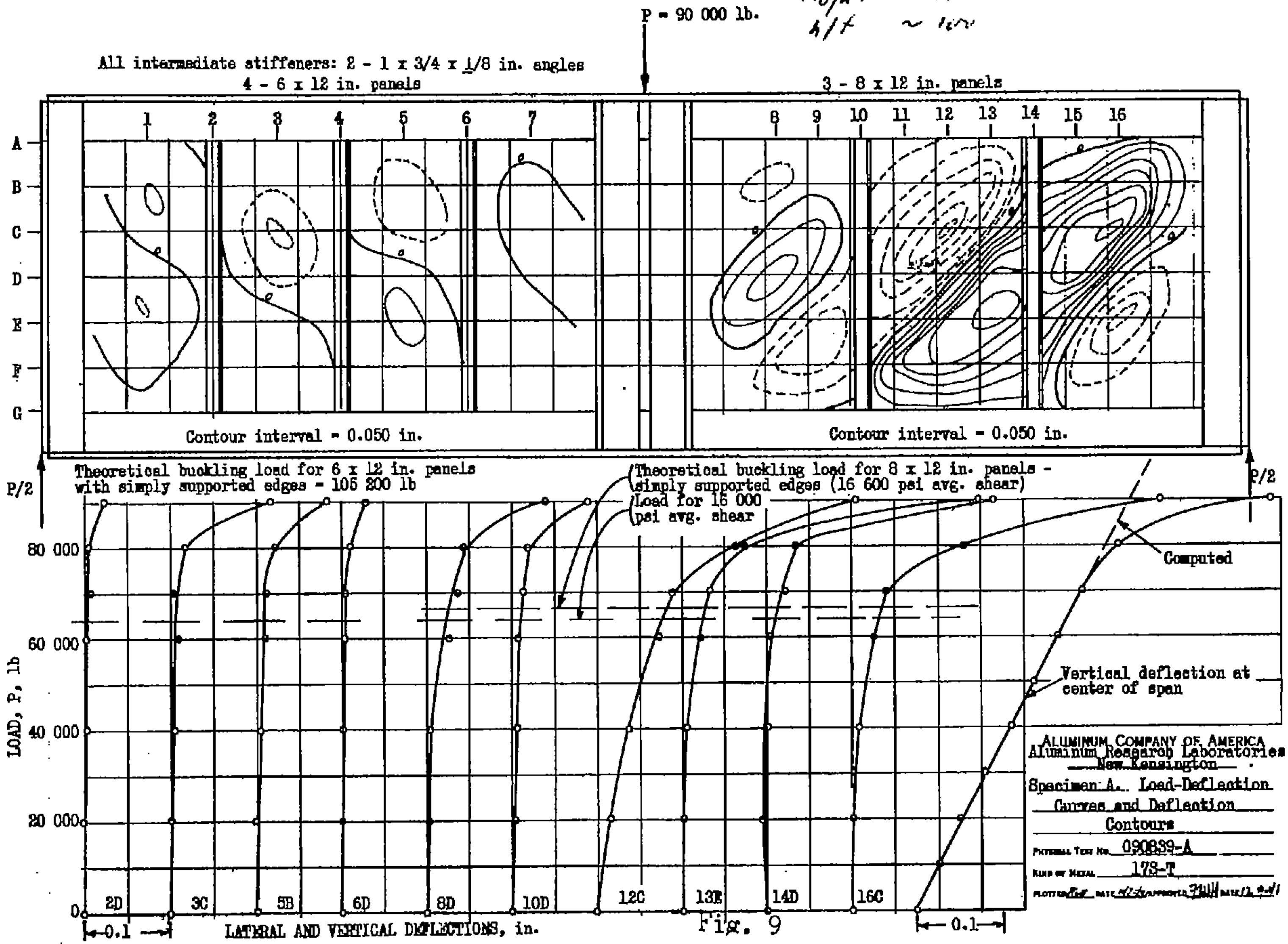


Fig. 9

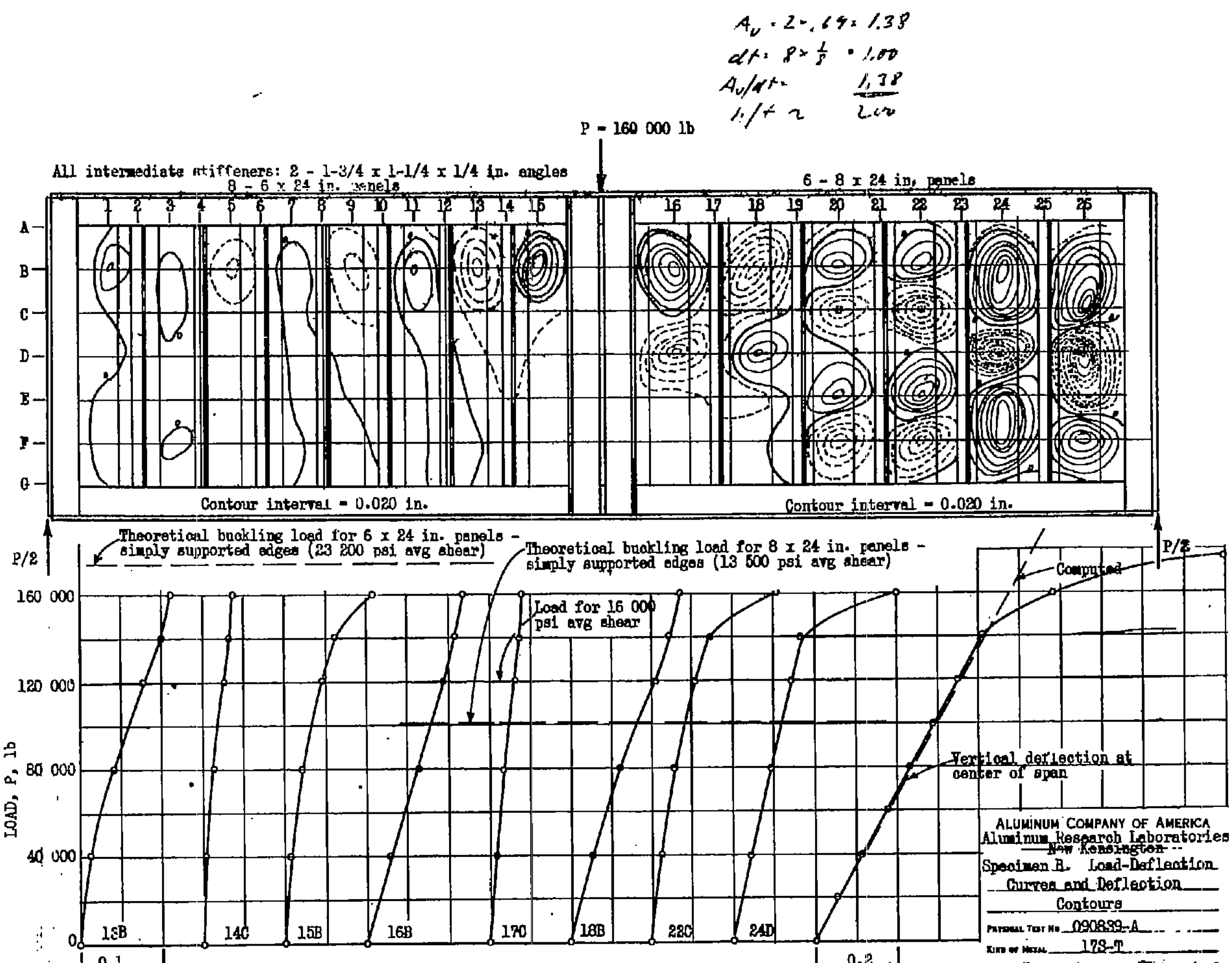
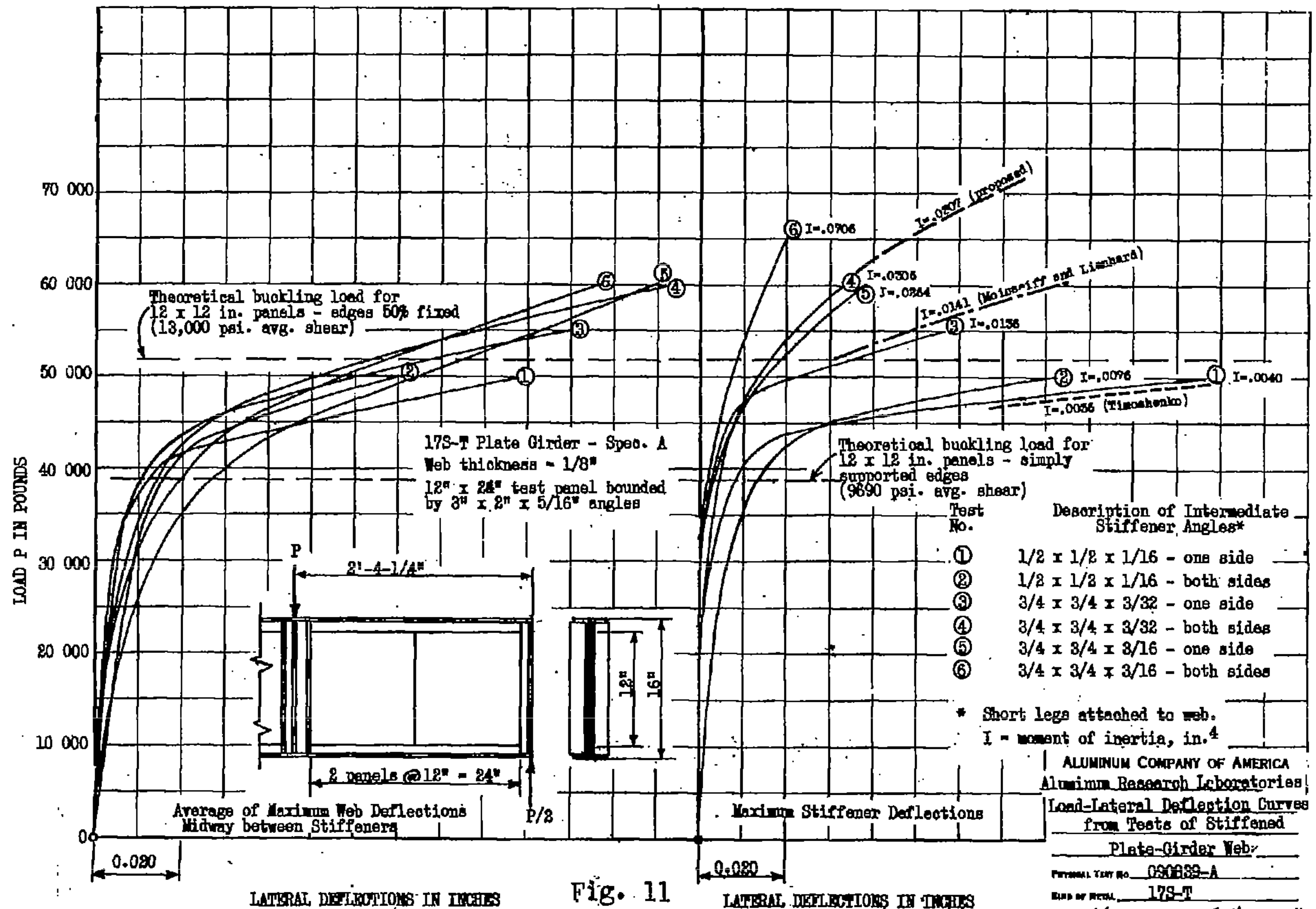
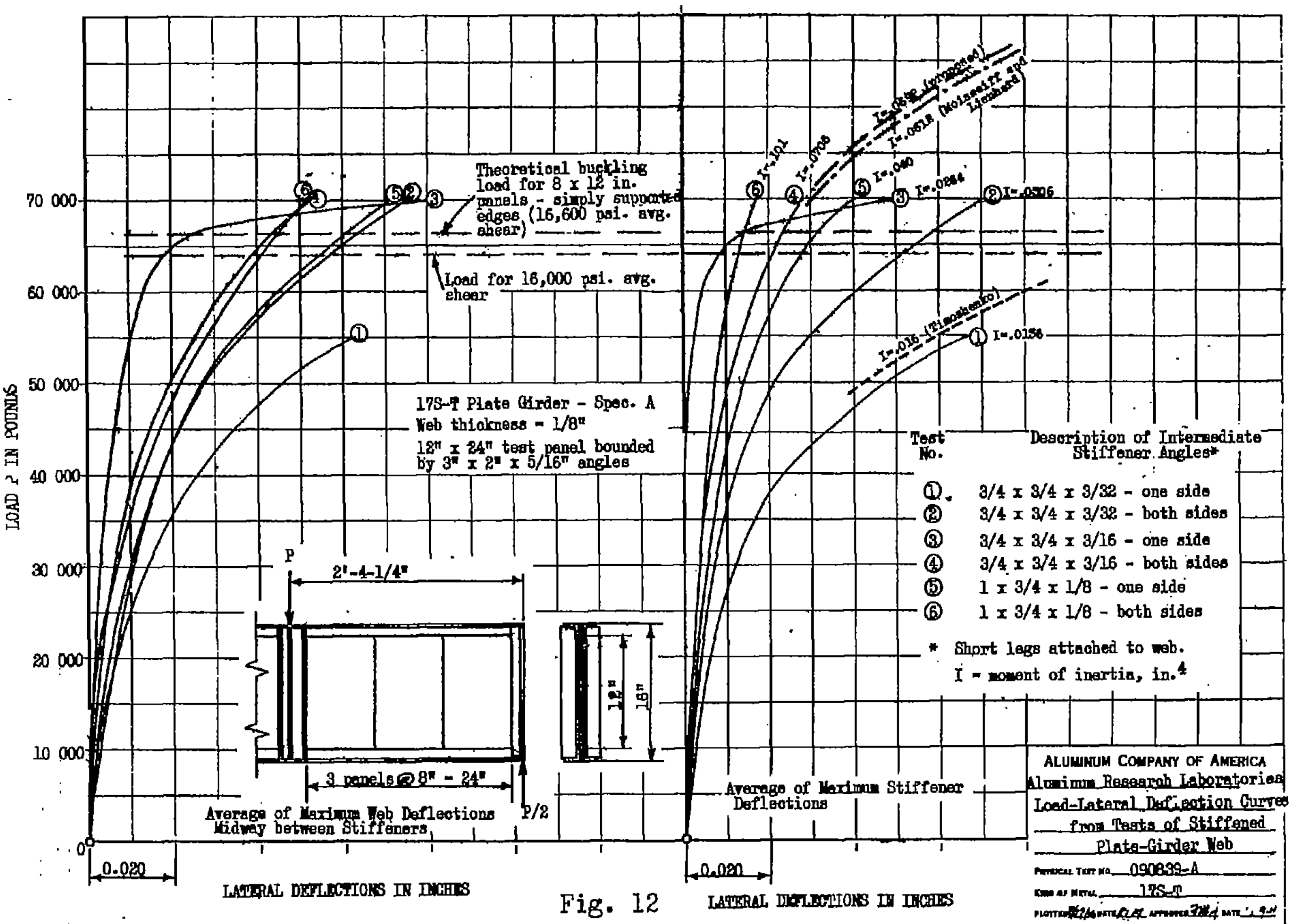

Av = 2 × .69 = 1.38
dt = 8 × 1/8 = 1.00
Av/dt = 1.38
1/t = 2.00
P = 160 000 lb
All intermediate stiffeners: 2 - 1-3/4 x 1-1/4 x 1/4 in. angles
8 - 6 x 24 in. panels
6 - 8 x 24 in. panels
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26
A B C D E F G
Contour interval = 0.020 in.
Contour interval = 0.020 in.
Theoretical buckling load for 6 x 24 in. panels - simply supported edges (23 200 psi avg shear)
Theoretical buckling load for 8 x 24 in. panels - simply supported edges (13 500 psi avg shear)
Load for 16 000 psi avg shear
Computed
Vertical deflection at center of span
P/2
P/2
LOAD, P, lb
0
40 000
80 000
120 000
160 000
13B
14C
15B
16B
17C
18B
22C
24D
0.1
0.2
LATERAL AND VERTICAL DEFLECTIONS, in.
ALUMINUM COMPANY OF AMERICA
Aluminum Research Laboratories
New Kensington
Specimen B. Load-Deflection Curves and Deflection Contours
Physical Test No. 090839-A
Kind of Metal 17S-T

Fig. 10



LOAD P IN POUNDS
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
Theoretical buckling load for
12 x 12 in. panels - edges 50% fixed
(13,000 psi. avg. shear)
17S-T Plate Girder - Spec. A
Web thickness - 1/8"
12" x 24" test panel bounded
by 3" x 2" x 5/16" angles
P
2'-4-1/4"
2 panels @12" = 24"
12"
16"
P/2
Average of Maximum Web Deflections
Midway between Stiffeners
0.020
LATERAL DEFLECTIONS IN INCHES
I=.0706
I=.0207 (proposed)
I=.0305
I=.0264
I=.0141 (Moissieff and Lienhard)
I=.0136
I=.0076
I=.0040
I=.0036 (Timoshenko)
Theoretical buckling load for
12 x 12 in. panels - simply
supported edges
(9690 psi. avg. shear)
Test No.
Description of Intermediate Stiffener Angles*
① 1/2 x 1/2 x 1/16 - one side
② 1/2 x 1/2 x 1/16 - both sides
③ 3/4 x 3/4 x 3/32 - one side
④ 3/4 x 3/4 x 3/32 - both sides
⑤ 3/4 x 3/4 x 3/16 - one side
⑥ 3/4 x 3/4 x 3/16 - both sides
* Short legs attached to web.
I = moment of inertia, in.4
Maximum Stiffener Deflections
0.020
LATERAL DEFLECTIONS IN INCHES
ALUMINUM COMPANY OF AMERICA
Aluminum Research Laboratories
Load-Lateral Deflection Curves
from Tests of Stiffened
Plate-Girder Web
090839-A
17S-T

Fig. 11



LOAD P IN POUNDS
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
Theoretical buckling load for 8 x 12 in. panels - simply supported edges (16,600 psi. avg. shear)
Load for 16,000 psi. avg. shear
17S-T Plate Girder - Spec. A
Web thickness = 1/8"
12" x 24" test panel bounded by 3" x 2" x 5/16" angles
P
2'-4-1/4"
3 panels @ 8" = 24"
P/2
12"
16"
Average of Maximum Web Deflections Midway between Stiffeners
0.020
LATERAL DEFLECTIONS IN INCHES
I=.101
I=.0706
I=.040
I=.0284
I=.0306
I=.0136
I=.016 (Timoshenko)
I=.0618 (Moissieff and Lienhard)
Average of Maximum Stiffener Deflections
0.020
LATERAL DEFLECTIONS IN INCHES
Test No.
Description of Intermediate Stiffener Angles*
① 3/4 x 3/4 x 3/32 - one side
② 3/4 x 3/4 x 3/32 - both sides
③ 3/4 x 3/4 x 3/16 - one side
④ 3/4 x 3/4 x 3/16 - both sides
⑤ 1 x 3/4 x 1/8 - one side
⑥ 1 x 3/4 x 1/8 - both sides
* Short legs attached to web.
I = moment of inertia, in.4
ALUMINUM COMPANY OF AMERICA
Aluminum Research Laboratories
Load-Lateral Deflection Curves from Tests of Stiffened Plate-Girder Web
Physical Test No. 090839-A
Kind of Metal 17S-T

Fig. 12

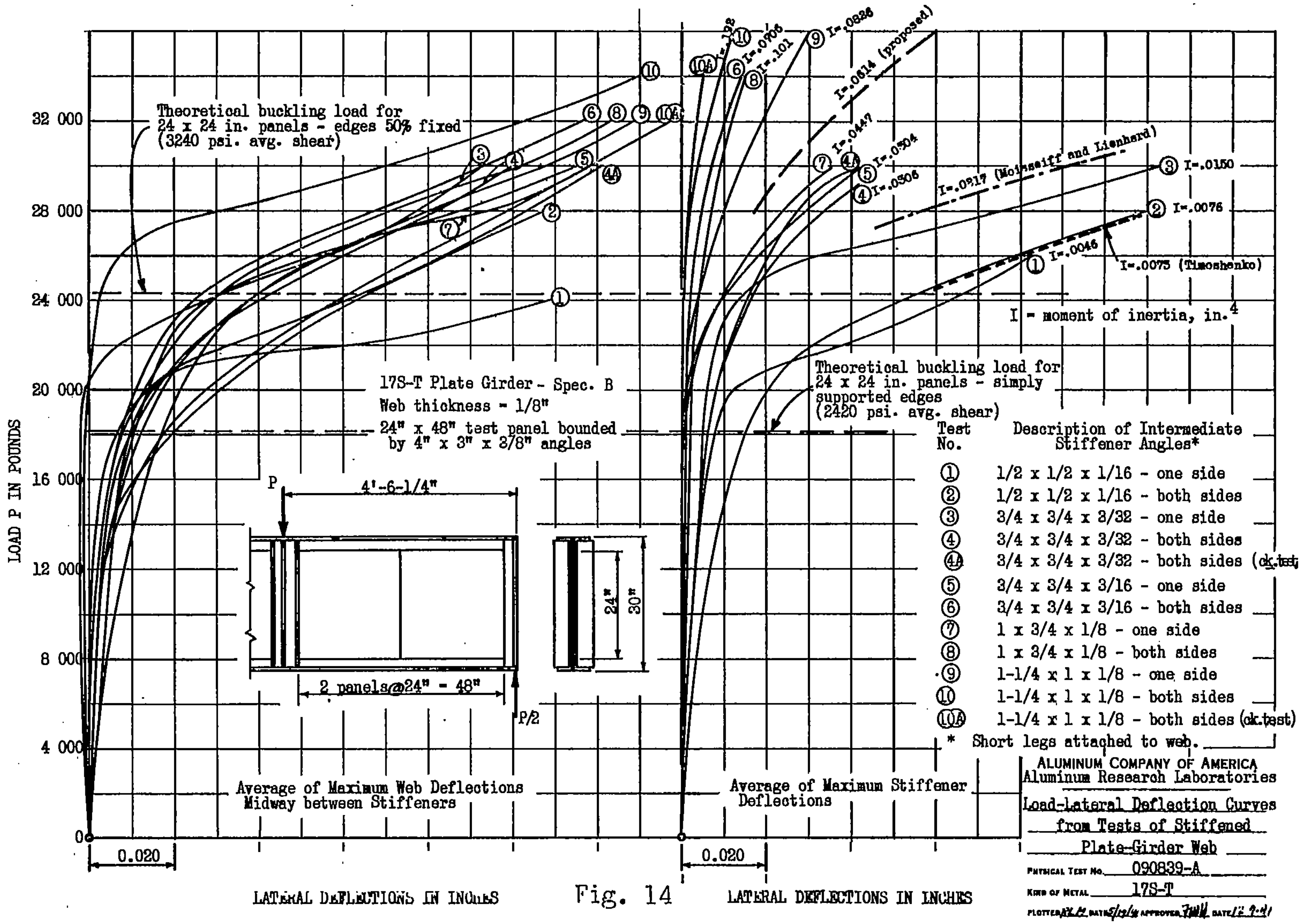

Theoretical buckling load for 24 x 24 in. panels - edges 50% fixed (3240 psi. avg. shear)
Theoretical buckling load for 24 x 24 in. panels - simply supported edges (2420 psi. avg. shear)
17S-T Plate Girder - Spec. B
Web thickness = 1/8"
24" x 48" test panel bounded by 4" x 3" x 3/8" angles
P
4'-6-1/4"
2 panels@24" = 48"
P/2
24"
30"
LOAD P IN POUNDS
32 000
28 000
24 000
20 000
16 000
12 000
8 000
4 000
0
0.020
Average of Maximum Web Deflections Midway between Stiffeners
LATERAL DEFLECTIONS IN INCHES
Average of Maximum Stiffener Deflections
LATERAL DEFLECTIONS IN INCHES
I=.0814 (proposed)
I=.0317 (Moisseiff and Lienhard)
I=.0075 (Timoshenko)
I = moment of inertia, in.^{4}
I=.0150
I=.0076
I=.0046
I=.0306
I=.0504
I=.0447
I=.0828
I=.101
I=.0706
I=.122
Test No.
Description of Intermediate Stiffener Angles*
1 1/2 x 1/2 x 1/16 - one side
2 1/2 x 1/2 x 1/16 - both sides
3 3/4 x 3/4 x 3/32 - one side
4 3/4 x 3/4 x 3/32 - both sides
4A 3/4 x 3/4 x 3/32 - both sides (ck.test)
5 3/4 x 3/4 x 3/16 - one side
6 3/4 x 3/4 x 3/16 - both sides
7 1 x 3/4 x 1/8 - one side
8 1 x 3/4 x 1/8 - both sides
9 1-1/4 x 1 x 1/8 - one side
10 1-1/4 x 1 x 1/8 - both sides
10A 1-1/4 x 1 x 1/8 - both sides (ck.test)
* Short legs attached to web.
ALUMINUM COMPANY OF AMERICA
Aluminum Research Laboratories
Load-Lateral Deflection Curves from Tests of Stiffened Plate-Girder Web
PHYSICAL TEST NO. 090839-A
KIND OF METAL 17S-T

Fig. 14

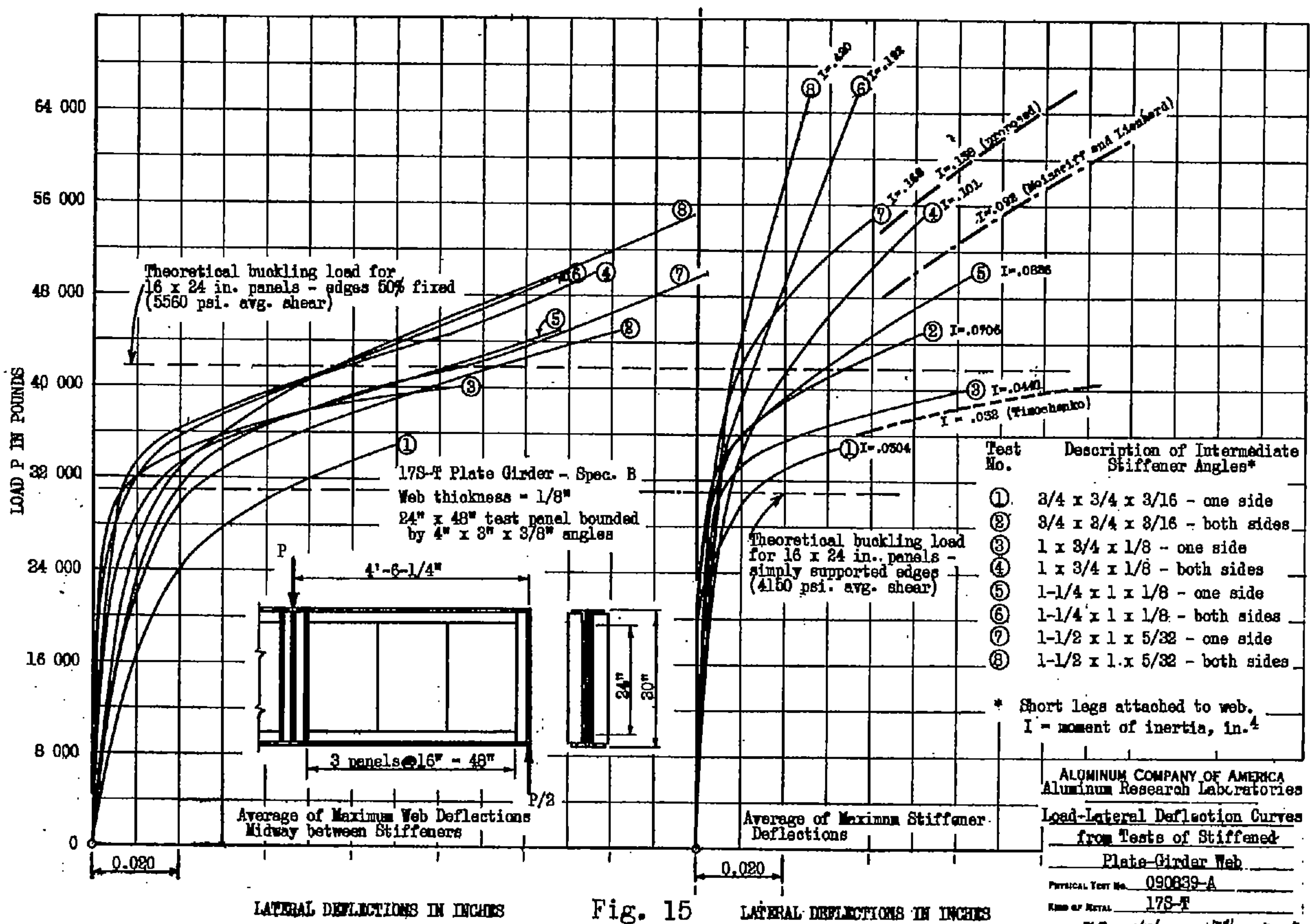

LOAD P IN POUNDS
64 000
56 000
48 000
40 000
32 000
24 000
16 000
8 000
0
Theoretical buckling load for 16 x 24 in. panels - edges 50% fixed (5560 psi. avg. shear)
17S-T Plate Girder - Spec. B
Web thickness = 1/8"
24" x 48" test panel bounded by 4" x 3" x 3/8" angles
P
4'-6-1/4"
24"
30"
3 panels @ 16" = 48"
P/2
Average of Maximum Web Deflections Midway between Stiffeners
0.020
LATERAL DEFLECTIONS IN INCHES
I=.430
I=.182
I=.168
I=.159 (proposed)
I=.101
I=.092 (Moisseiff and Lienhard)
I=.0686
I=.0706
I=.0440
I=.033 (Timoshenko)
I=.0304
Theoretical buckling load for 16 x 24 in. panels - simply supported edges (4150 psi. avg. shear)
Average of Maximum Stiffener Deflections
0.020
LATERAL DEFLECTIONS IN INCHES
Test No.
Description of Intermediate Stiffener Angles*
1 3/4 x 3/4 x 3/16 - one side
2 3/4 x 3/4 x 3/16 - both sides
3 1 x 3/4 x 1/8 - one side
4 1 x 3/4 x 1/8 - both sides
5 1-1/4 x 1 x 1/8 - one side
6 1-1/4 x 1 x 1/8 - both sides
7 1-1/2 x 1 x 5/32 - one side
8 1-1/2 x 1 x 5/32 - both sides
* Short legs attached to web.
I = moment of inertia, in.4
ALUMINUM COMPANY OF AMERICA
Aluminum Research Laboratories
Load-Lateral Deflection Curves from Tests of Stiffened Plate-Girder Web
Physical Test No. 090839-A
Kind of Metal 17S-T

Fig. 15

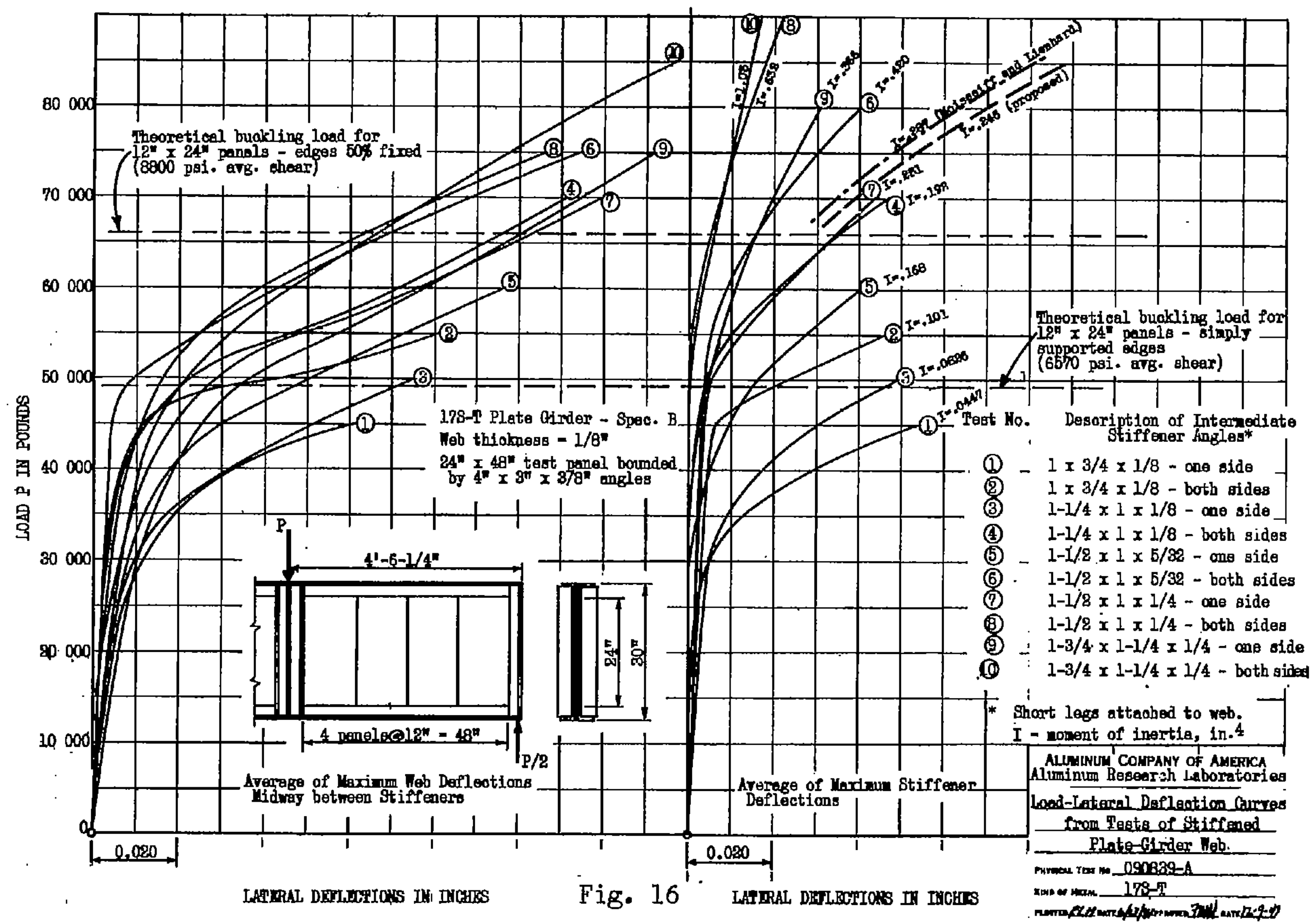
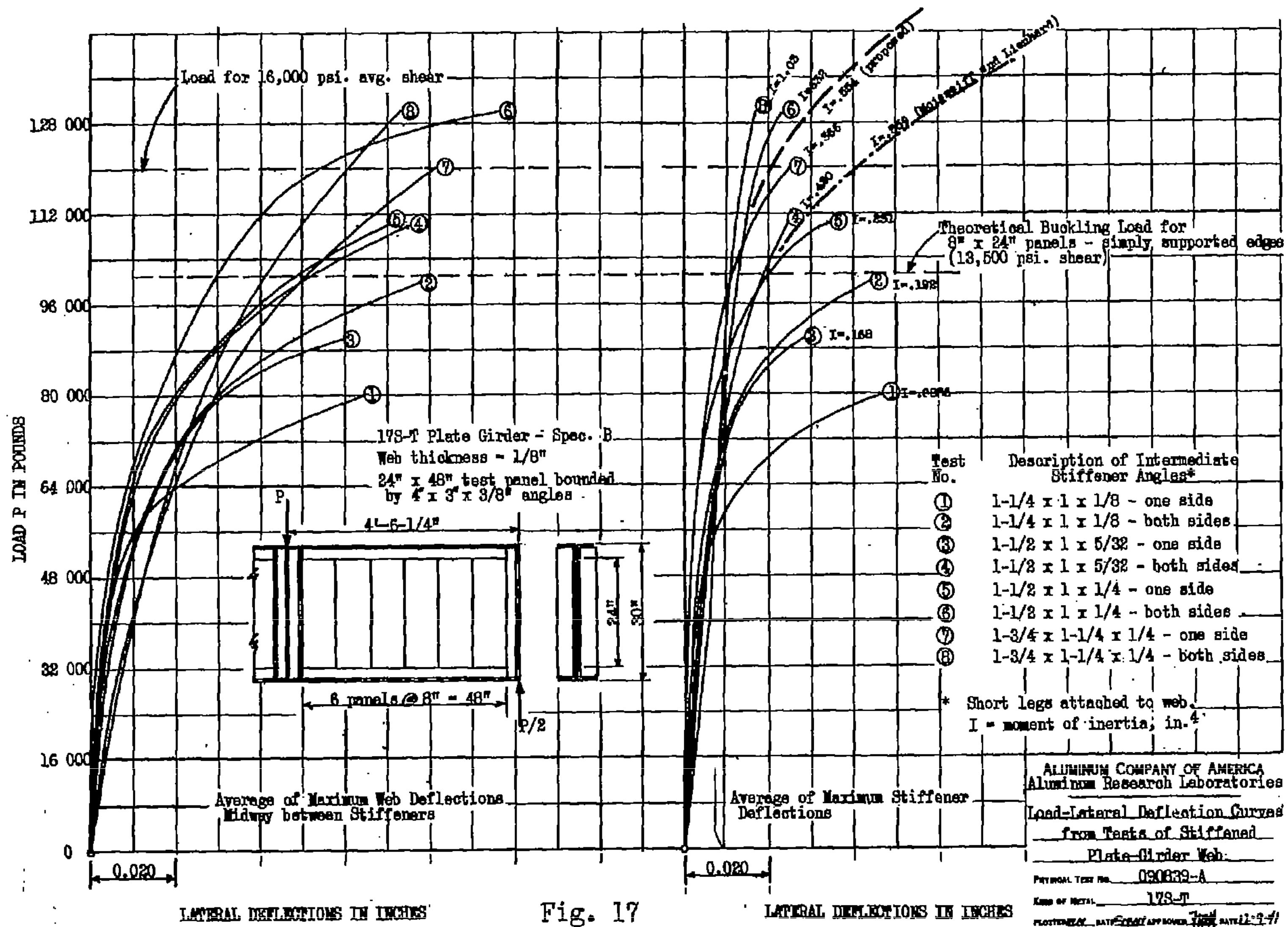

Theoretical buckling load for 12" x 24" panels - edges 50% fixed (8800 psi. avg. shear)
Theoretical buckling load for 12" x 24" panels - simply supported edges (6570 psi. avg. shear)
LOAD P IN POUNDS
80 000
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
0.020
17S-T Plate Girder - Spec. B
Web thickness = 1/8"
24" x 48" test panel bounded by 4" x 3" x 3/8" angles
P
4'-6-1/4"
24"
30"
4 panels@12" = 48"
P/2
Average of Maximum Web Deflections Midway between Stiffeners
Average of Maximum Stiffener Deflections
LATERAL DEFLECTIONS IN INCHES
LATERAL DEFLECTIONS IN INCHES
I=1.08
I=.658
I=.368
I=.490
I=.337 (Moissieff and Lienhard)
I=.246 (proposed)
I=.221
I=.199
I=.168
I=.101
I=.0626
I=.0447
Test No.
Description of Intermediate Stiffener Angles*
1 - 1 x 3/4 x 1/8 - one side
2 - 1 x 3/4 x 1/8 - both sides
3 - 1-1/4 x 1 x 1/8 - one side
4 - 1-1/4 x 1 x 1/8 - both sides
5 - 1-1/2 x 1 x 5/32 - one side
6 - 1-1/2 x 1 x 5/32 - both sides
7 - 1-1/2 x 1 x 1/4 - one side
8 - 1-1/2 x 1 x 1/4 - both sides
9 - 1-3/4 x 1-1/4 x 1/4 - one side
10 - 1-3/4 x 1-1/4 x 1/4 - both sides
* Short legs attached to web.
I - moment of inertia, in.4
ALUMINUM COMPANY OF AMERICA
Aluminum Research Laboratories
Load-Lateral Deflection Curves from Tests of Stiffened Plate-Girder Web.
PHYSICAL TEST No. 090839-A
KIND OF METAL 17S-T

Fig. 16



Load for 16,000 psi. avg. shear
Theoretical Buckling Load for 8" x 24" panels - simply supported edges (13,500 psi. shear)
LOAD P IN POUNDS
128 000
112 000
96 000
80 000
64 000
48 000
32 000
16 000
0
17S-T Plate Girder - Spec. B
Web thickness = 1/8"
24" x 48" test panel bounded by 4" x 3" x 3/8" angles
P
4'-6-1/4"
6 panels @ 8" = 48"
P/2
24"
30"
Average of Maximum Web Deflections Midway between Stiffeners
Average of Maximum Stiffener Deflections
0.020
0.020
LATERAL DEFLECTIONS IN INCHES
LATERAL DEFLECTIONS IN INCHES
I=1.03
I=.832
I=.554 (proposed)
I=.365
I=.280 (Moissieff and Lienhard)
I=.430
I=.281
I=.192
I=.168
I=.0876
Test No.
Description of Intermediate Stiffener Angles*
1 1-1/4 x 1 x 1/8 - one side
2 1-1/4 x 1 x 1/8 - both sides
3 1-1/2 x 1 x 5/32 - one side
4 1-1/2 x 1 x 5/32 - both sides
5 1-1/2 x 1 x 1/4 - one side
6 1-1/2 x 1 x 1/4 - both sides
7 1-3/4 x 1-1/4 x 1/4 - one side
8 1-3/4 x 1-1/4 x 1/4 - both sides
* Short legs attached to web.
I = moment of inertia, in.4
ALUMINUM COMPANY OF AMERICA
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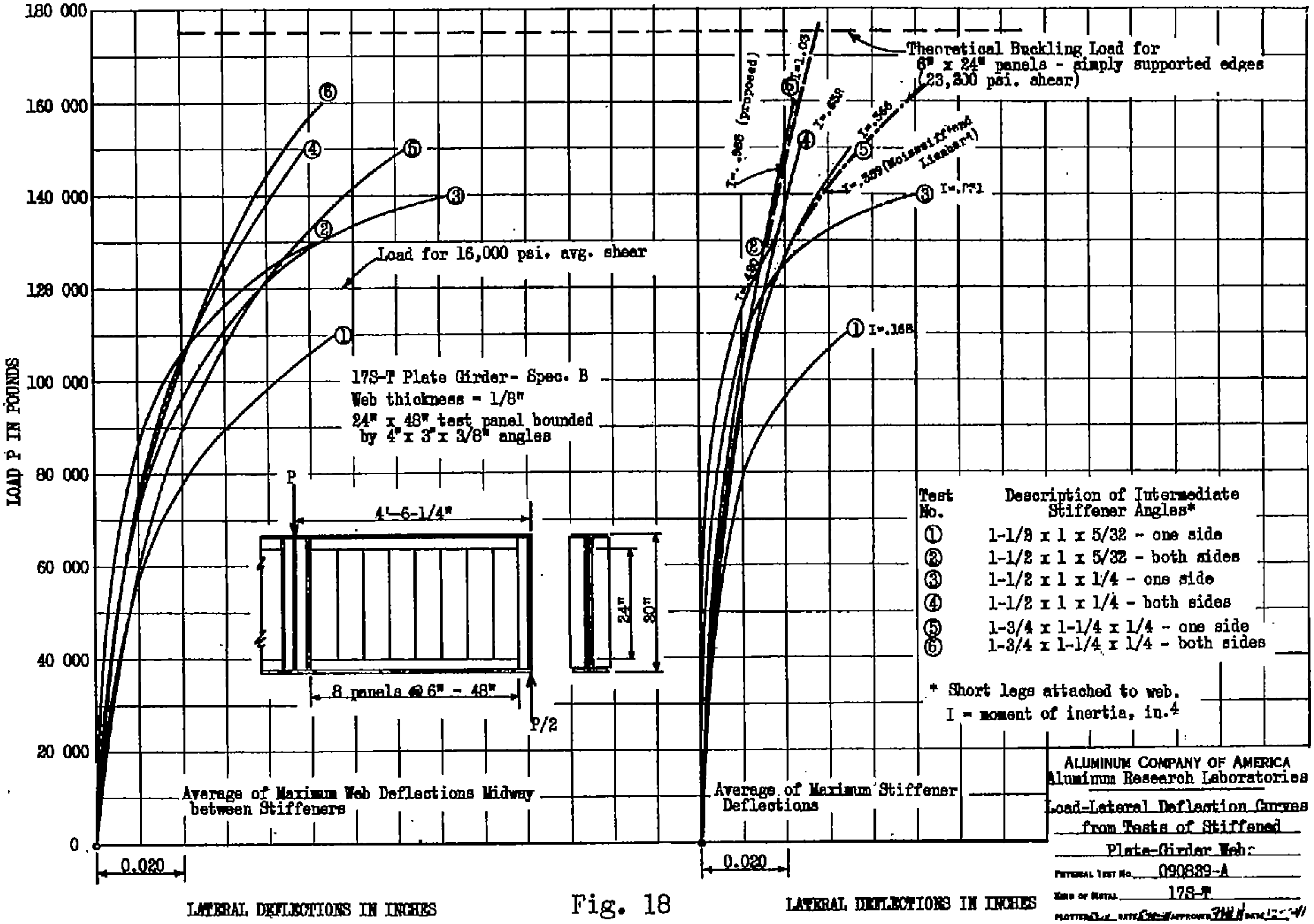
Load-Lateral Deflection Curves from Tests of Stiffened Plate-Girder Web.
PHYSICAL TEST No. 090839-A
KIND OF METAL 17S-T

Fig. 17



180 000
160 000
140 000
120 000
100 000
80 000
60 000
40 000
20 000
0
LOAD P IN POUNDS
Theoretical Buckling Load for 6" x 24" panels - simply supported edges (23,300 psi. shear)
Load for 16,000 psi. avg. shear
17S-T Plate Girder- Spec. B
Web thickness = 1/8"
24" x 48" test panel bounded by 4" x 3" x 3/8" angles
P
4'-6-1/4"
8 panels @ 6" = 48"
P/2
24"
30"
Test No.
Description of Intermediate Stiffener Angles*
① 1-1/2 x 1 x 5/32 - one side
② 1-1/2 x 1 x 5/32 - both sides
③ 1-1/2 x 1 x 1/4 - one side
④ 1-1/2 x 1 x 1/4 - both sides
⑤ 1-3/4 x 1-1/4 x 1/4 - one side
⑥ 1-3/4 x 1-1/4 x 1/4 - both sides
* Short legs attached to web.
I = moment of inertia, in.4
I=.168
I=.271
I=.389 (Moissieff and Lienhard)
I=.566
I=.638
I=.965 (proposed)
I=1.03
Average of Maximum Web Deflections Midway between Stiffeners
Average of Maximum Stiffener Deflections
0.020
0.020
LATERAL DEFLECTIONS IN INCHES
LATERAL DEFLECTIONS IN INCHES
ALUMINUM COMPANY OF AMERICA
Aluminum Research Laboratories
Load-Lateral Deflection Curves from Tests of Stiffened Plate-Girder Web
PHYSICAL TEST No. 090839-A
17S-T

Fig. 18

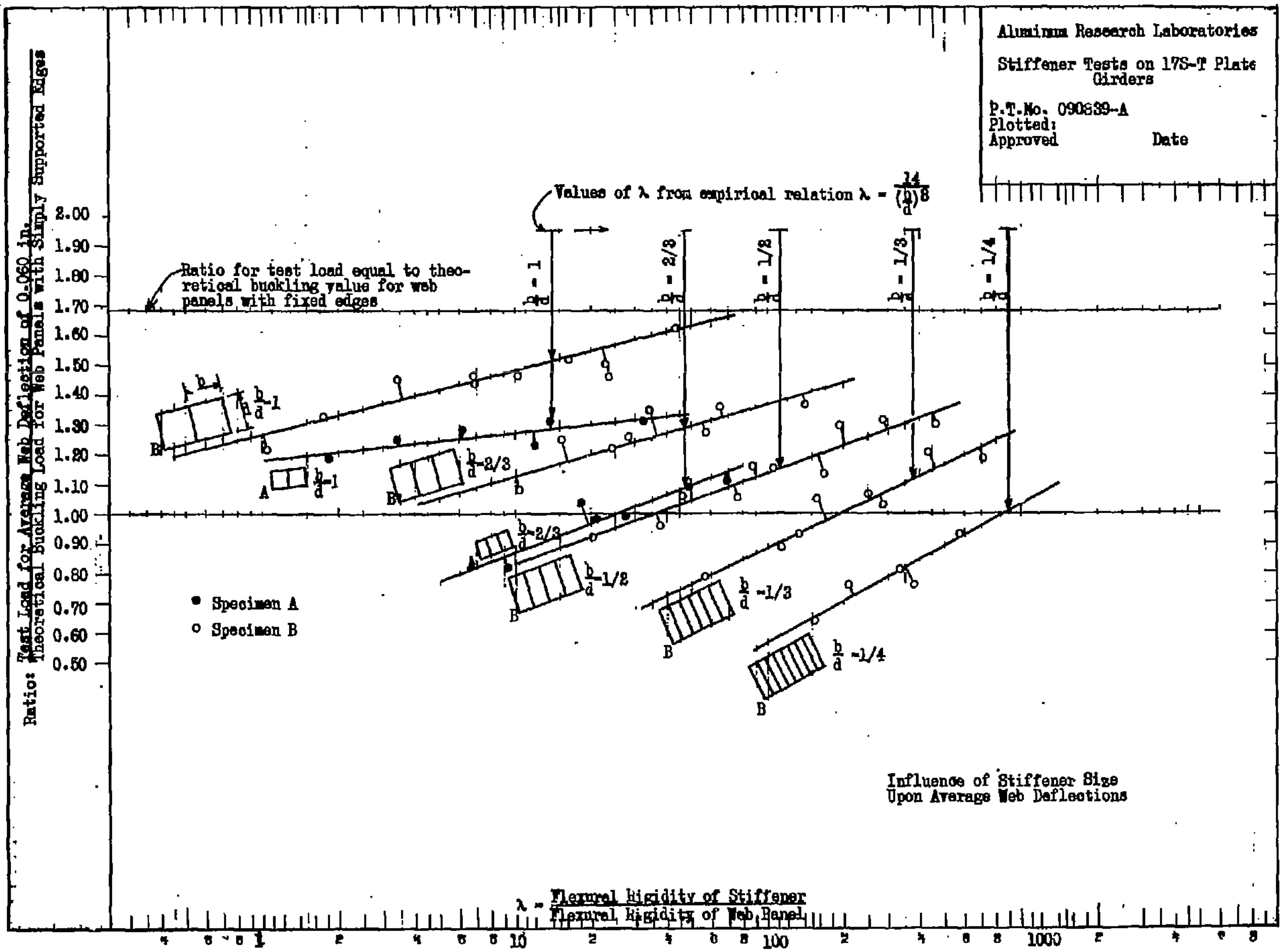
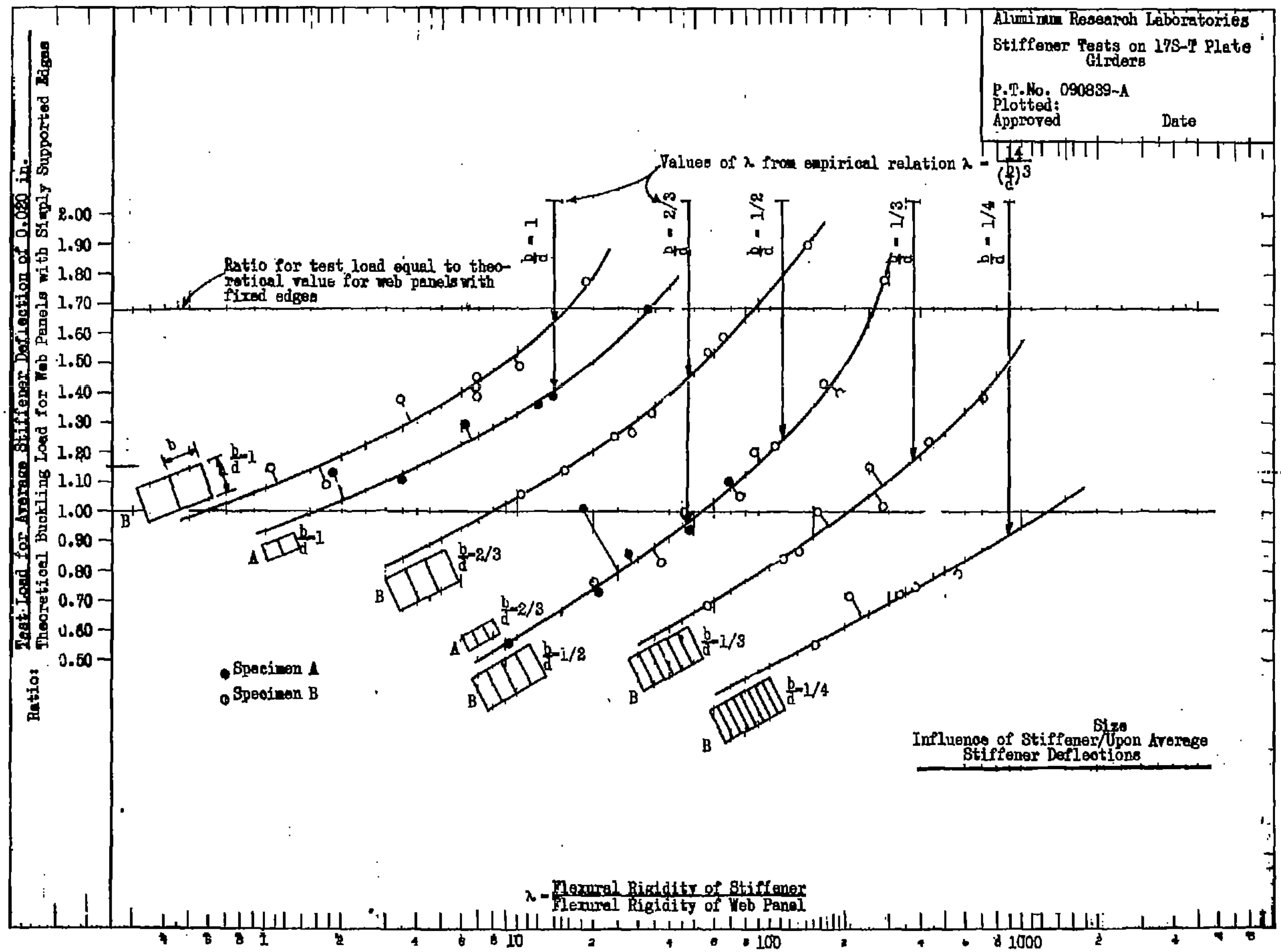

Aluminum Research Laboratories
Stiffener Tests on 17S-T Plate Girders
P.T.No. 090839-A
Plotted:
Approved
Date
Values of λ from empirical relation λ = 14/(b/d)^3
b/d = 1
b/d = 2/3
b/d = 1/2
b/d = 1/3
b/d = 1/4
Ratio for test load equal to theoretical buckling value for web panels with fixed edges
Ratio: Test Load for Average Web Deflection of 0.060 in. / Theoretical Buckling Load for Web Panels with Simply Supported Edges
2.00
1.90
1.80
1.70
1.60
1.50
1.40
1.30
1.20
1.10
1.00
0.90
0.80
0.70
0.60
0.50
● Specimen A
○ Specimen B
A
B
Influence of Stiffener Size Upon Average Web Deflections
λ = Flexural Rigidity of Stiffener / Flexural Rigidity of Web Panel
1
10
100
1000

Fig. 19



Aluminum Research Laboratories
Stiffener Tests on 17S-T Plate Girders
P.T.No. 090839-A
Plotted:
Approved
Date
Values of λ from empirical relation λ = 14/(b/d)^3
Ratio for test load equal to theoretical value for web panels with fixed edges
Ratio: Test Load for Average Stiffener Deflection of 0.020 in. / Theoretical Buckling Load for Web Panels with Simply Supported Edges
b/d = 1
b/d = 2/3
b/d = 1/2
b/d = 1/3
b/d = 1/4
Specimen A
Specimen B
Influence of Stiffener Size Upon Average Stiffener Deflections
λ = Flexural Rigidity of Stiffener / Flexural Rigidity of Web Panel

Fig. 20

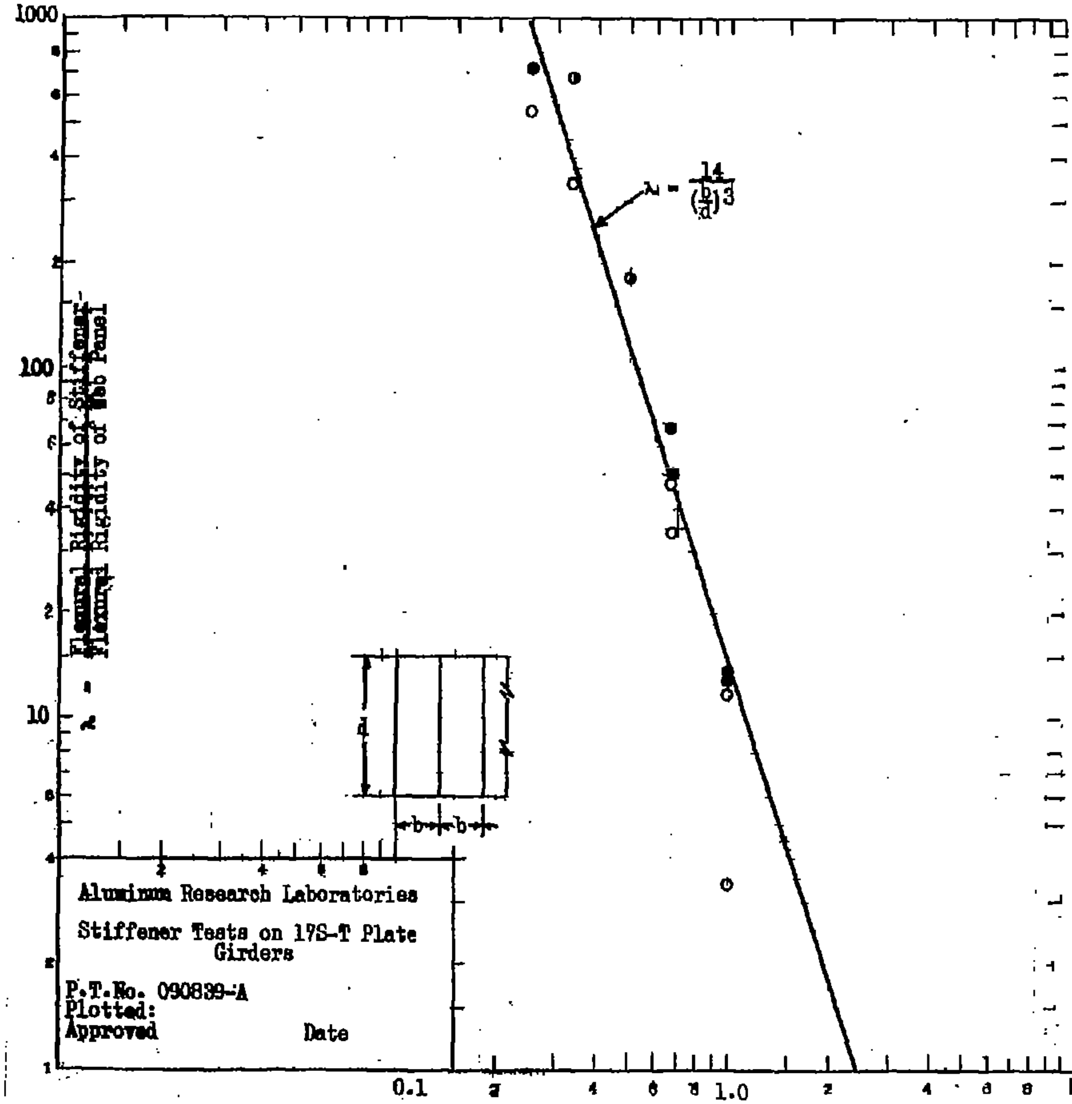


Fig. 21 $\frac{b}{d} = \frac{\text{Stiffener Spacing}}{\text{Panel Depth}}$

Proposed Empirical Relation Between Ratios of Flexural Rigidity λ and Proportions of Web Panel

$$\lambda = \frac{12EI(1-\mu^2)}{Ebt^3}$$

where I = moment of inertia of stiffener, in.^{4}
E = modulus of elasticity, psi.,
b = stiffener spacing, in.,
t = web thickness, in.,
and μ = Poisson's ratio = 1/3

● Values of λ obtained from tests in which average stiffener deflections were approximately 25 per cent of average web deflections for assumed critical loads.*

o Values of λ obtained from tests in which average stiffener deflections were approximately 0.020 in. for assumed critical loads.*

* Assumed critical loads equal to theoretical buckling values for web panels with edges 50 per cent fixed, (34 per cent greater than buckling loads for simply supported edges) but not exceeding average shear stress of 16,000 psi.

Figure 22.- Specimen A after failure of stiffener connections and fracture of web under load of 93,300 pounds.

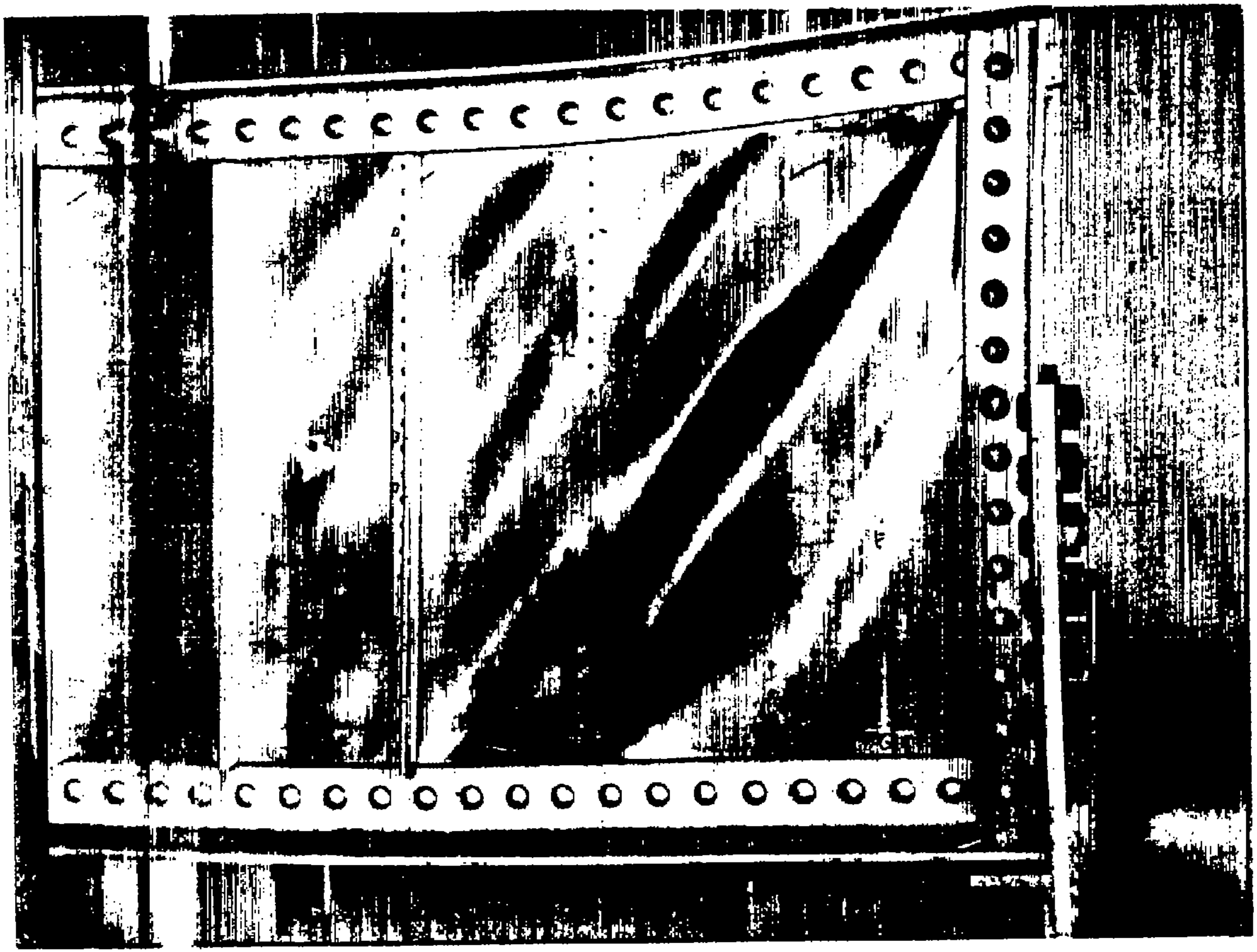
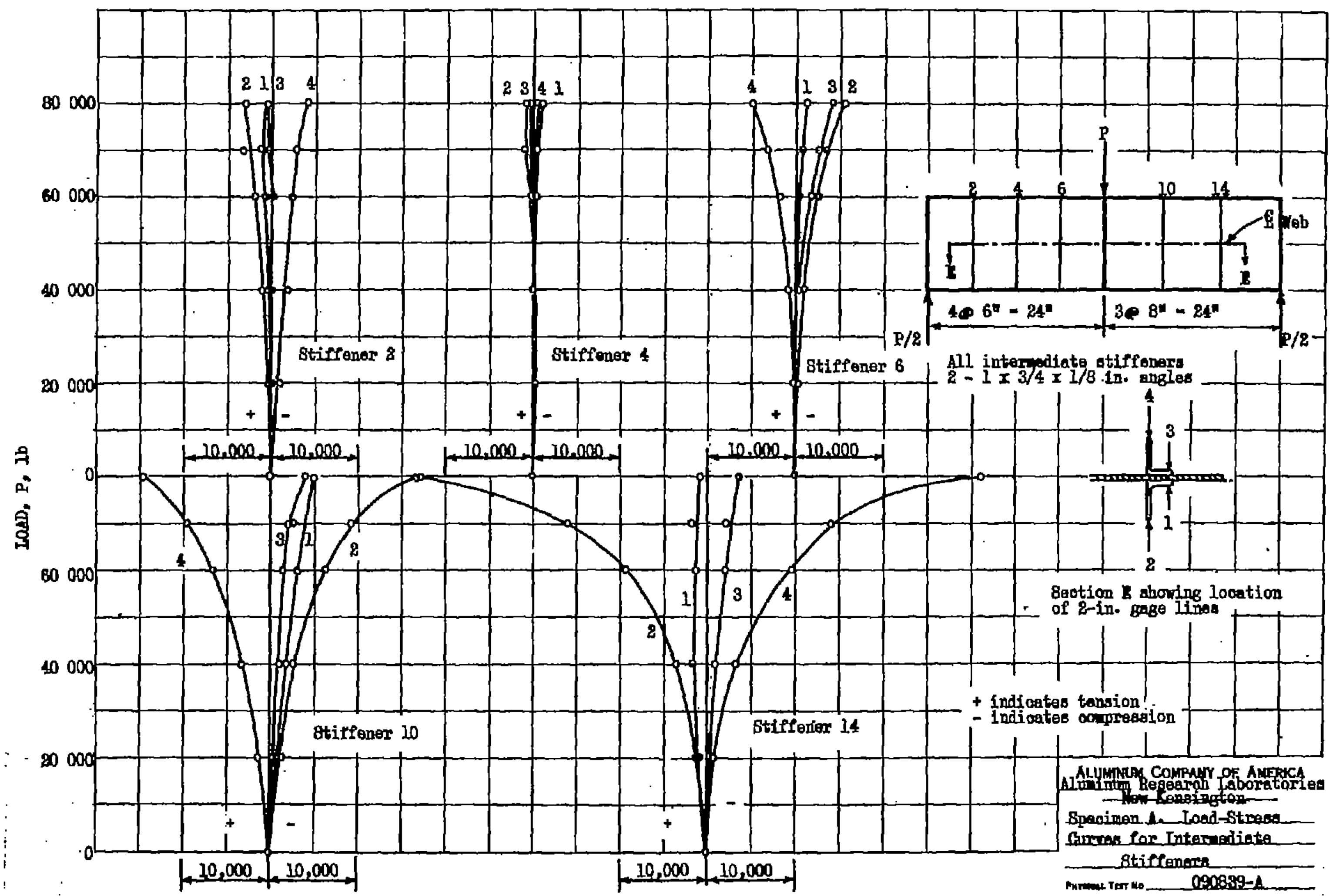


Figure 23.- Specimen B after failure of stiffener connections and collapse of web under load of 191,500 pounds.


LOAD, P, lb
STRESSES, psi
Stiffener 2
Stiffener 4
Stiffener 6
Stiffener 10
Stiffener 14
10,000
+
-
All intermediate stiffeners
2 - 1 x 3/4 x 1/8 in. angles
4@ 6" = 24"
3@ 8" = 24"
P
P/2
Web
Section E showing location of 2-in. gage lines
+ indicates tension
- indicates compression
Aluminum Company of America
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New Kensington
Specimen A. Load-Stress Curves for Intermediate Stiffeners
090839-A
17S-T

Fig. 24

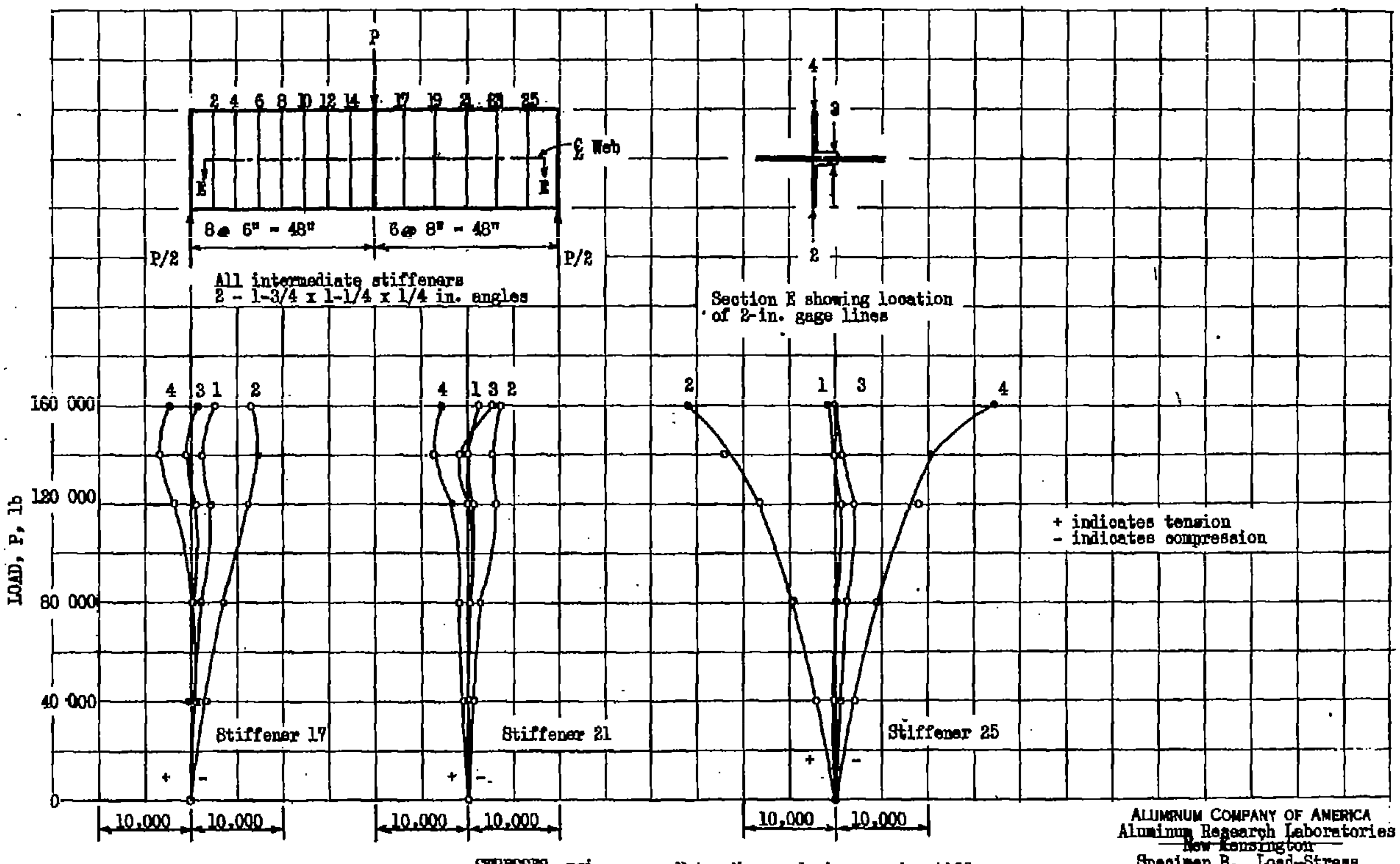

Fig. 25

Note: Measured stresses in stiffeners 2, 6, 10 and 14 of negligible magnitude

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New Kensington
Specimen B. Load-Stress
Curves for Intermediate
Stiffeners
Physical Test No. 090839-A
Kind of Metal 17S-T

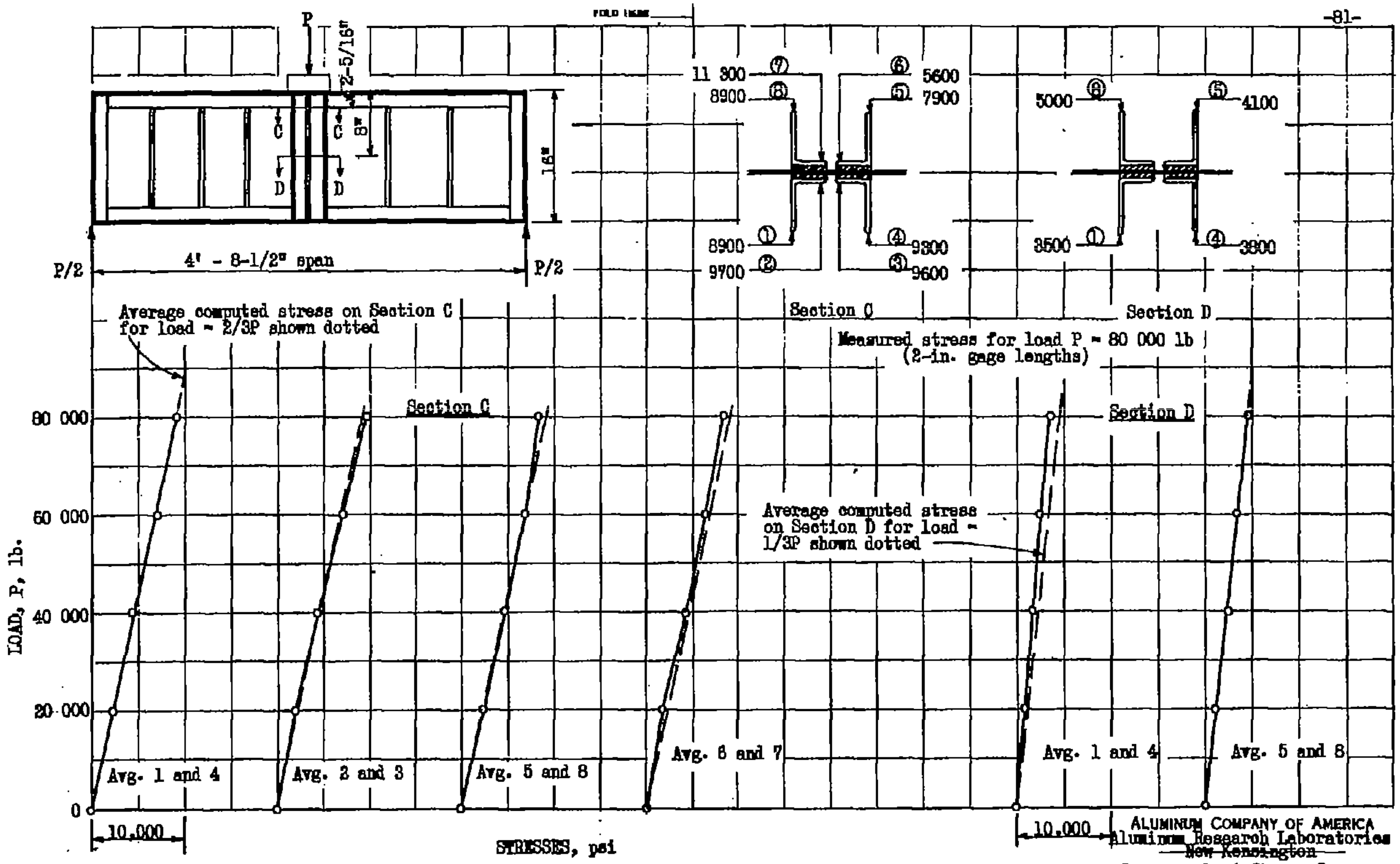

Fig. 26

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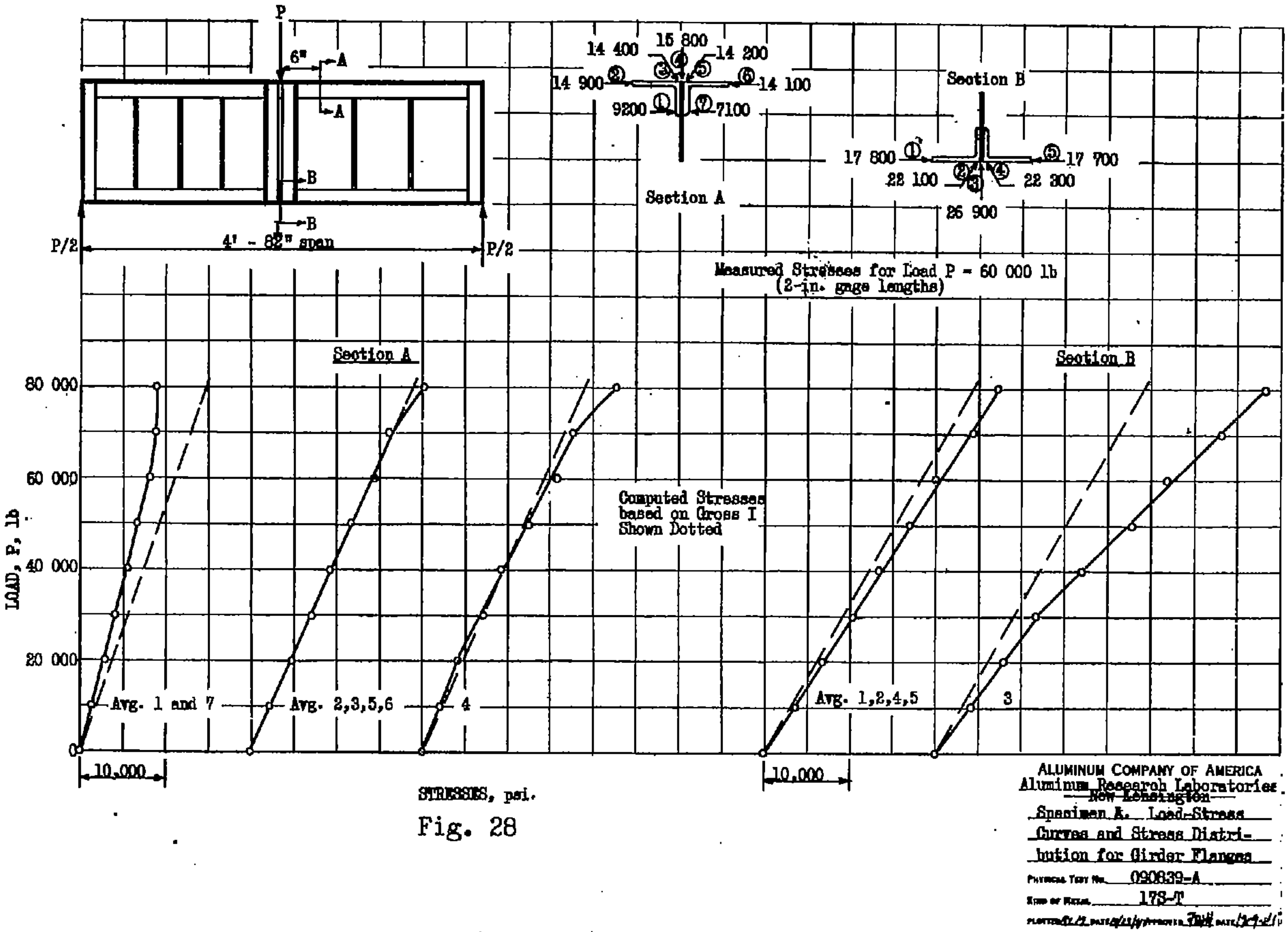
New Kensington

Spec. A. Load-Stress Curves and Stress Distribution for Load-Bearing Stiffeners

PHYSICAL TEST No. 090839-A

KIND OF METAL 17S-T



P
6"
A
A
B
B
P/2
4' - 8½" span
P/2
14 400
15 800
14 200
14 900
14 100
9200
7100
Section A
Section B
17 800
17 700
22 100
22 300
26 900
Measured Stresses for Load P = 60 000 lb
(2-in. gage lengths)
Section A
Section B
80 000
60 000
40 000
20 000
0
LOAD, P, lb
Computed Stresses based on Gross I Shown Dotted
Avg. 1 and 7
Avg. 2,3,5,6
4
Avg. 1,2,4,5
3
10,000
10,000
STRESSES, psi.
ALUMINUM COMPANY OF AMERICA
Aluminum Research Laboratories
New Kensington
Specimen A. Load-Stress Curves and Stress Distribution for Girder Flanges
Physical Test No. 090839-A
Kind of Metal 17S-T

Fig. 28

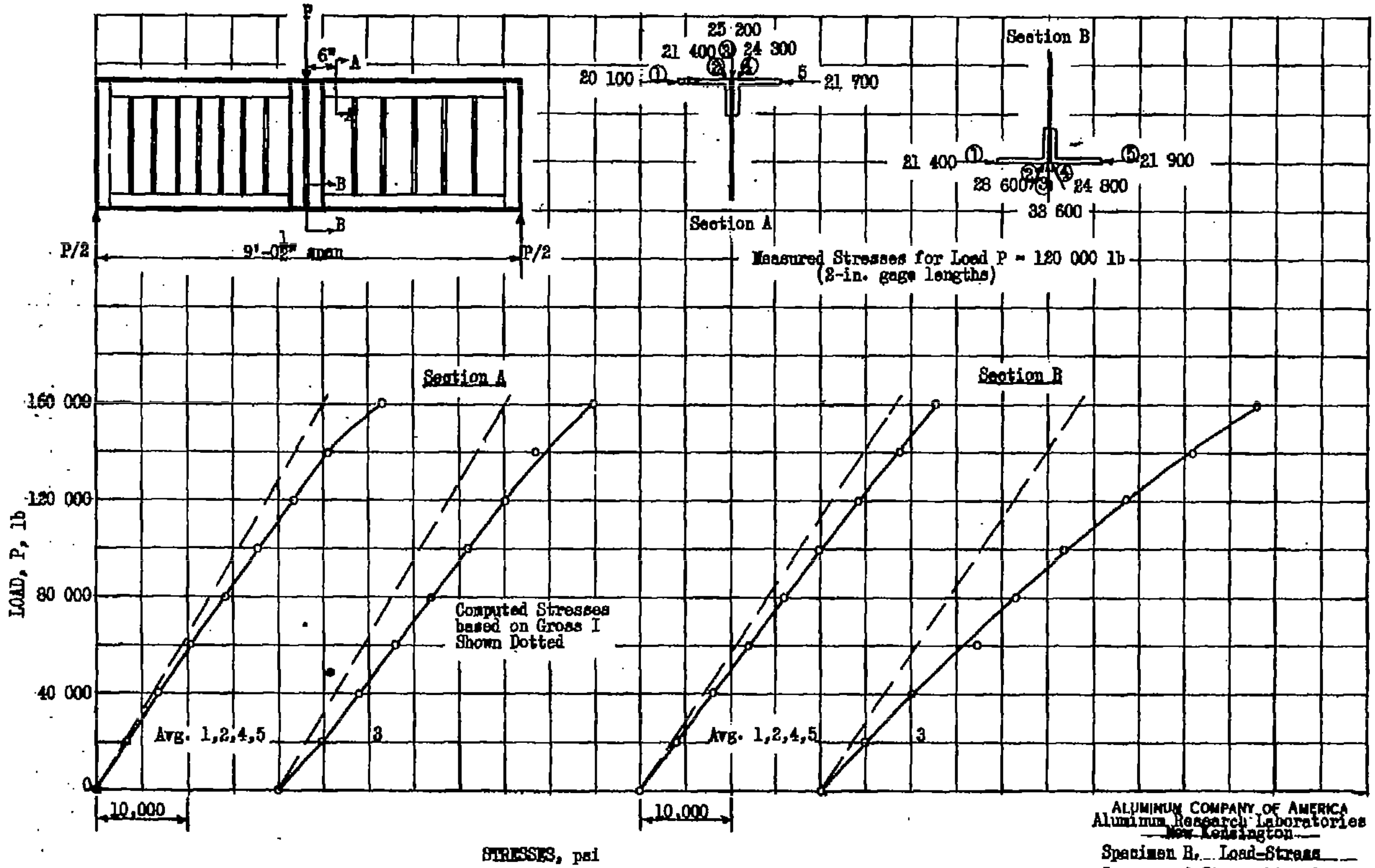


Fig. 29

ALUMINUM COMPANY OF AMERICA
Aluminum Research Laboratories
New Kensington

Specimen R, Load-Stress Curves and Stress Distribution for Girder Flanges

Physical Test No. 090839-A

Kind of Metal 17S-T